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OF THE

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VOL. 74 DECEMBER, 1948 No. 10

TECHNICAL PAPERS

AND

DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

EFFECT OF DRIVING PILES INTO SOFT CLAY

BY A. E. CUMMINGS,[1] M. ASCE, G. O. KERKHOFF,[2] AND R. B. PECK,[3] ASSOC. M. ASCE

SYNOPSIS

This paper describes the results of field and laboratory tests made to determine the extent of the disturbance produced by driving piles into a soft, glacial, lake clay deposit in Detroit, Mich. The results indicate that the natural structure of the clay was appreciably disturbed close to the piles, but that the amount of disturbance did not approach that of complete remolding in the laboratory. At a distance of about two times the diameter of the pile, the clay within a pile group was disturbed only a negligible amount. The process of consolidation initiated by the disturbance was not of a type that would cause settlement of the ground surface.

INTRODUCTION

Publication in 1932[4] of the results of unconfined and confined compression tests on undisturbed and completely remolded samples of plastic clays from the Laurentian Valley, Canada, and from the subsoil of Boston, Mass., brought to the attention of engineers for the first time the great increase in compressibility which some clays undergo when disturbed. As a consequence of this discovery, it was pointed out that driving piles into soft clay deposits might cause sufficient disturbance to produce appreciable settlement of the surface of the deposits merely under the weight of the soil itself, and that the presence of the piles might prove detrimental rather than beneficial. On the basis of the information available at that time, it was suggested that the clay immediately surrounding the pile would be completely remolded to a distance of one half of the diameter of the pile, and that to a distance of one and one-half diameters it would be sufficiently disturbed to cause a rather large increase in its com-

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1949.**

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[4] "The Structure of Clay and Its Importance in Foundation Engineering," by A. Casagrande, *Journal*, Boston Soc. of Civ. Engrs., April, 1932.

pressibility. On the assumption that the presence of the piles had no influence on the settlement of the surrounding clay, other than to produce the disturbance, it was computed that the settlements of certain buildings in Boston might actually be twice as great if they were founded on friction piles than if they were supported directly by the clay.

Although A. Casagrande, M. ASCE, cautioned that, in order to get conclusive data concerning the remolding effect of pile driving, it would be necessary to conduct elaborate field investigations, the tentative conclusions which he drew have been widely accepted. Some engineers even believe that pile driving in soft clays is always detrimental and should be avoided.

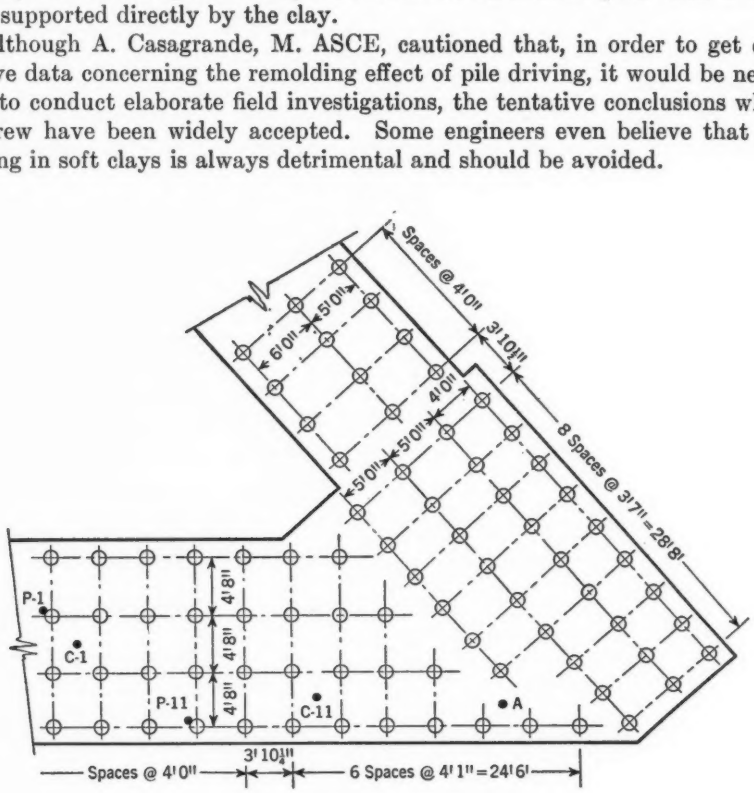

FIG. 1.—LOCATION OF TEST BORINGS

In 1944 an opportunity presented itself to obtain quantitative information concerning the remolding effect of pile driving when the Detroit Industrial Expressway was under construction. At the site of the grade separation between the expressway and the main line of the Michigan Central Railroad, the soil for a depth of about 80 ft is the typical soft blue glacial lake clay deposit found throughout the River Rouge (Mich.) area. It was decided to establish the abutments and center pier of the structure on cast-in-place concrete piles driven through the soft clay to bearing on the hardpan. The diameter of the piles increased from 10 in. at the point to 16 in. at the butt, the footing layout calling for piles at approximately 4½-ft centers. At this spacing, the ground surface between the piles heaved about 2 ft during the driving operations.

Before construction was started, the natural water contents and the stress-strain characteristics of the clay were determined by laboratory tests on soil

samples obtained from a number of borings. After the pile driving was completed, additional borings were made to determine the values of these same properties for some samples taken close to the piles and for other samples taken within the pile group at the greatest possible distance from the surrounding piles. Two pairs of borings were made, one pair about 3 weeks after the piles were driven, and the other about 11 months after pile driving. For convenience, the first pair will be referred to as the 1-month borings and the second as the 11-month borings. One boring of each pair was made adjacent to a pile and the other in the center of a group. The location of one of the original borings and of the four additional ones with respect to the piles is shown in Fig. 1. The original boring is designated by the symbol, A. The 1-month and 11-month borings adjacent to a pile are designated P-1 and P-11, respectively. Those in the center of a group of surrounding piles are denoted as C-1 and C-11, respectively. The 1-month borings were made before the pile cap was poured; the 11-month borings were made through sleeves cast in the concrete pile cap.

General Soil Conditions

A composite vertical section through the piles and the subsoil is shown in Fig. 2, which is based on the results of six borings made at various locations

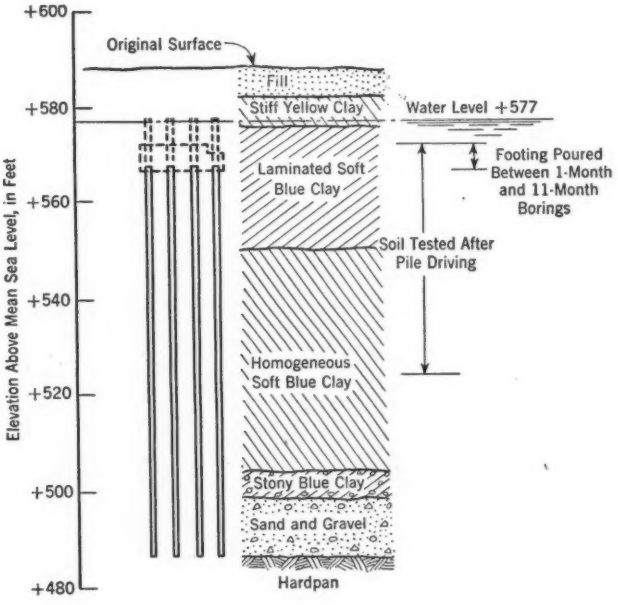

Fig. 2.—Composite Vertical Section of Soil in Pile Area

on the site before any piles were driven. These borings indicated that the soil was essentially uniform in its properties in horizontal directions over the entire area to be occupied by the structure.

The soft clay is divided at an elevation of approximately 550 ft into upper and lower regions with rather different characteristics. This fact is shown in Fig. 3, in which the variation of the natural water content, liquid limit, and plastic limit is plotted as a function of depth. Above the elevation of 550 ft the soft clay has a laminated structure consisting of alternating layers of silty clay and highly plastic clay, commonly only a few hundredths of an inch in thickness. The natural water content of the laminated clay is extremely variable and similar variation occurs in the values of the liquid and plastic limits. In their undisturbed state, the samples had an unconfined compressive strength approximately eight times that of the same samples when remolded in the laboratory at their natural water content. Therefore, the upper part of the deposit may be classified as a clay sensitive to disturbance.

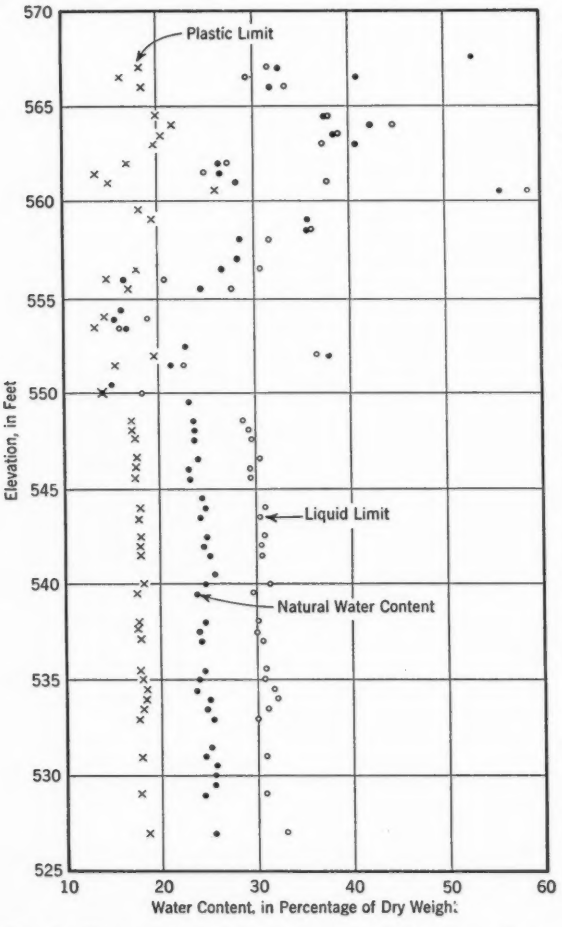

FIG. 3.—NATURAL WATER CONTENT, PLASTIC LIMIT, AND LIQUID LIMIT AT BORING A BEFORE PILE DRIVING

Below the 550-ft elevation the natural water content and the Atterberg limits of the clay are remarkably uniform. There is a slight tendency for both the water content and the liquid limit to increase with increasing depth, but there are no erratic variations between any of the values. The clay is homogeneous in appearance except for the fact that it contains an appreciable number of small shale pebbles up to $\frac{1}{4}$ in. in diameter. The unconfined compressive strength of undisturbed samples is only about twice that of the material when completely remolded at the same water content. Therefore, the clay in the lower part of the deposit may be classified as considerably less sensitive to disturbance.

The nature of the disturbance due to pile driving in the stratified clays in this region is illustrated in Fig. 4. After a temporary pile trestle had served

its purpose, the clay surrounding the piles was removed by excavation. Fig. 4 shows the distortion of the clay surrounding one of the piles at a depth of about 12 ft below the ground surface.

Method of Sampling and Testing

All the samples were taken in steel tubing having a 2-in. external diameter and a wall thickness of $\frac{1}{16}$ in. The tubes were $2\frac{1}{2}$ ft long and beveled to a sharp cutting edge. Samples were taken continuously through the clay layer using casing whenever necessary. Usually the casing was driven after two tube samples had been recovered. The tubing was forced into the ground under static pressure and was not driven.

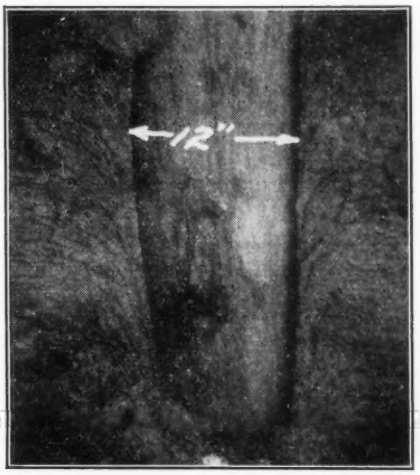

Fig. 4.—Clay Distortion Around Pile of Temporary Trestle

Boring A was made from the original ground surface before the piles were driven. The samples were practically undistorted (Fig. 5). On the basis of this evidence, it was considered satisfactory to proceed with the investigation by using 2-in. diameter samples.

Each tube was sealed in the field and shipped to the laboratory, where it was cut into 6-in. sections. The cylinders of soil were then removed and tested, each specimen being subjected first to an unconfined compression test in its intact state. It was then remolded to form another compression specimen

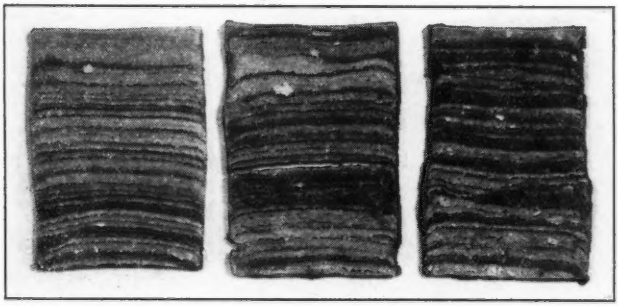

Fig. 5.—Samples from Boring A Before Pile Driving

and tested once more. A large part of the remolded sample was then used for the determination of the natural water content; the balance was used for determining the Atterberg limits. In the upper, laminated part of the soil deposit, a thin vertical section was cut from the undisturbed sample immediately

after the first compression test. This section was dried slowly in order to disclose the stratification, and later photographed.

Boring P-1 was made as close to a pile as possible. The pile itself was enclosed within a corrugated metal shell consisting of sections 12 ft long. Inasmuch as the bumping of the casing over the corrugations could be detected during the boring operations, it is certain that the samples from this boring were taken within 3 in. of the pile. Because of the high stresses that still existed in the clay after pile driving at the time this boring was made, the clay squeezed up into the casing as much as 1 ft before some of the samples could be obtained. This is an excessive amount of squeezing, and even on boring P-11 it was observed that there was a very noticeable squeezing of the clay into the boring casing. The samples next to the pile were therefore disturbed by the combined influence of pile driving and the flow of the soil into the casing. No such difficulty is encountered in normal boring operations in this area, and none was observed either in the borings made before pile driving or in the C-borings made between the piles after driving.

Results of Tests

The thin sections of samples taken adjacent to a pile 1 month after pile driving and 11 months after pile driving are shown in Figs. 6(*a*) and 6(*b*)

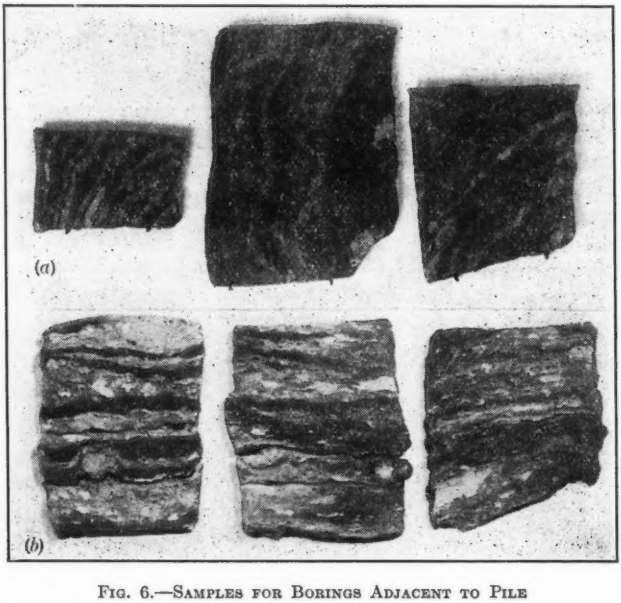

Fig. 6.—Samples for Borings Adjacent to Pile
(*a*) 1 Month After Pile Driving
(*b*) 11 Months After Pile Driving

respectively. Examination shows that the laminated structure of every sample was preserved. Although the layers were distorted, and in the case of the samples from boring P-1 severely distorted, they nevertheless retained their identity. Hence, it may be said that the disturbance by no means amounted to

complete remolding, a process by which all the structure and stratification of a sample are destroyed.

Inasmuch as the disturbance of the soil could not have increased by any conceivable method during the period between 1 month and 11 months after pile driving, it is reasonable to assume that the specimens from boring P-11 indicate most reliably the real distortion of the soil adjacent to the pile. These samples show much less distortion than those from boring P-1. The additional disturbance of the samples from boring P-1 can therefore be attributed exclusively to the squeezing of the clay into the casing during the sampling process.

Close examination of the specimens from boring P-11 suggests that the layers were compressed horizontally and that faults occurred. This would seem inevitable because of the displacement of the clay produced by the insertion of the piles. Nevertheless, the distortion is not excessive.

Samples taken from boring C-1, in the center of a surrounding group of piles, are shown in Fig. 7. The upper inch of the undisturbed sample (Fig.

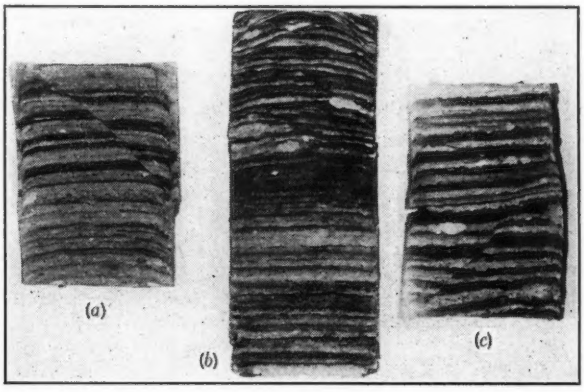

FIG. 7.—SAMPLES FROM CENTER OF PILE GROUP, 1 MONTH AFTER PILE DRIVING

7(*b*)) is more distorted than the remainder, and the laminations have a shape which indicates that the distortion was due to sampling rather than to the effects of pile driving. Figs. 7(*a*) and 7(*c*) show sections that were cut after the unconfined compression tests were performed. Therefore, they show the shear planes along which the specimens failed in the laboratory. It is apparent that none of the three specimens shown in Fig. 7 has experienced distortions more severe than those produced by the sampling processes in the undisturbed clay deposit before any piles were driven. In so far as distortion is concerned, the soil at the locations of C-1 and C-11 can be considered undisturbed.

The effect of pile driving on the soil adjacent to the pile and in the center of a pile group is shown quantitatively by the unconfined compressive strength diagram (Fig. 8) of samples from borings A, P-1, P-11, and C-1, and of the clay in its completely remolded state at the original water content. In Fig. 8, the data for boring P-1 indicate that in the upper, more sensitive part of the deposit, the compressive strength of the soil adjacent to the pile was decreased

by pile driving to a value intermediate between the completely remolded and the original strength. If the remolded strength is considered to be the lowest value to which the compressive strength can be reduced, it may be said that the pile driving reduced the compressive strength slightly more than half of

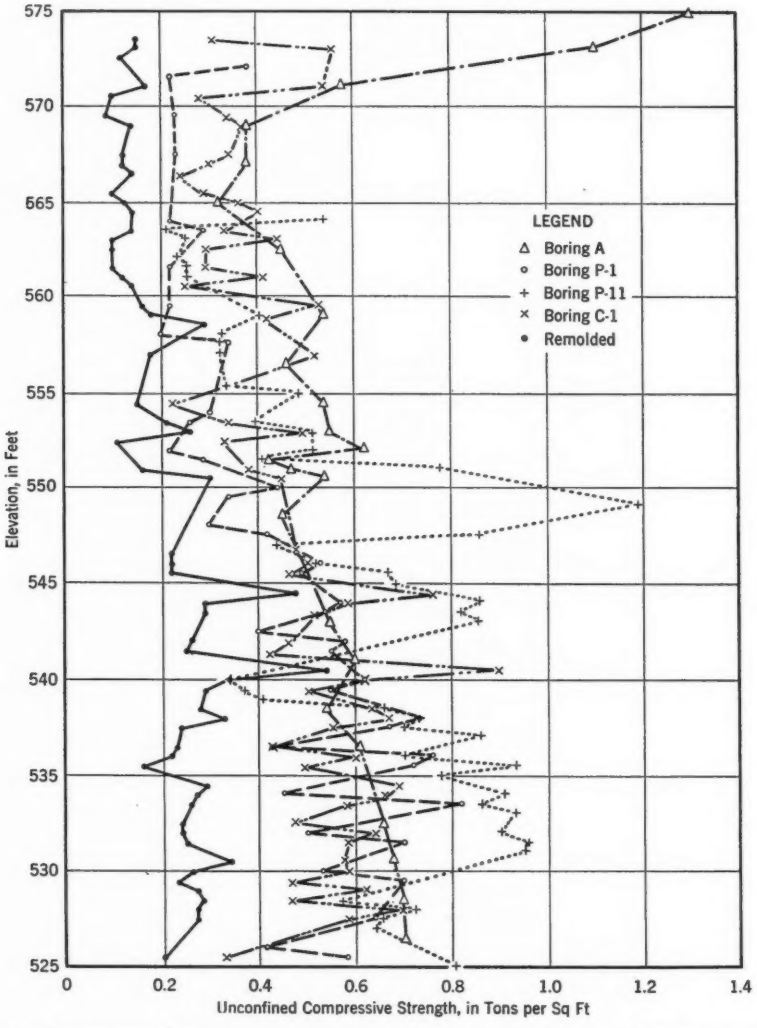

FIG. 8.—UNCONFINED COMPRESSIVE STRENGTHS OF SAMPLES FROM VARIOUS UNDISTURBED TEST BORINGS AND AS COMPLETELY REMOLDED

the maximum reduction possible. In the homogeneous clay layer constituting the lower part of the deposit the effect of pile driving appears to have been negligible. There is no essential difference between the average compressive strength of the soil before and after pile driving. The same information for

boring P-11 shows that the compressive strength of the upper, laminated part of the deposit has increased somewhat since boring P-1 was made, and in the lower part of the deposit the compressive strength of the soil is generally much greater than it was before the piles were driven. Similar information for boring C-1, taken in the center of the pile group, shows that there was a small difference in the unconfined compressive strength of the soil in the middle of the group before and after pile driving. A similar conclusion can be drawn concerning the samples from boring C-11.

Because of the erratic nature of the laminated part of the deposit, no significant information can be obtained from the water content data. However, the unusually uniform lower layer provides an exceptional opportunity

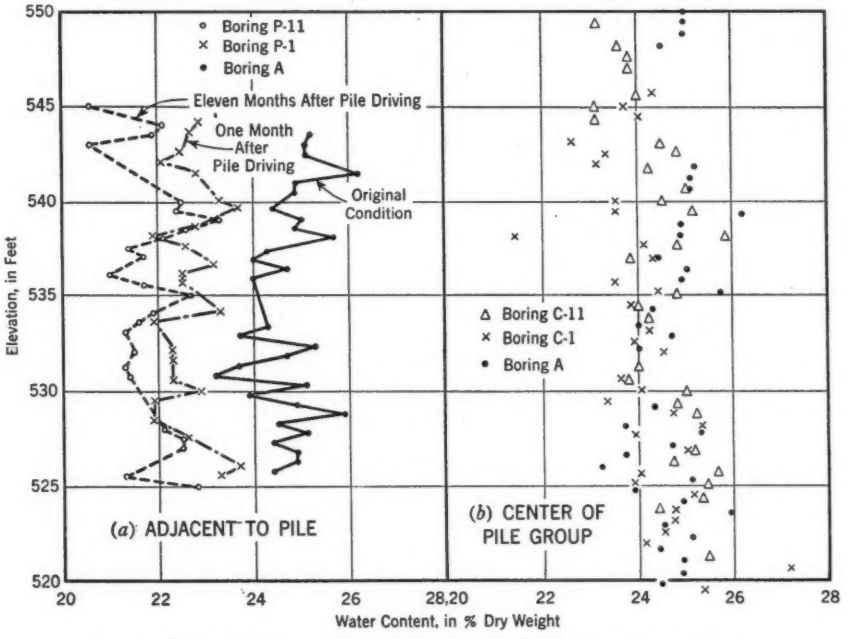

FIG. 9.—NATURAL WATER CONTENTS OF SAMPLES FROM VARIOUS TEST BORINGS

for studying the changes in water content which accompanied the changes in compressive strength previously described. Fig. 9 shows the variation in water content throughout the depth of this lower deposit at the three different times when borings were made. Originally the average value of the natural water content was 24.7%. A month after pile driving the water content adjacent to the pile (Fig. 9(*a*)) had decreased to 22.8%, and after 11 months it had decreased to 22.0%. Fig. 9(*a*) demonstrates clearly that the decrease in the average value of the water content was much greater than the statistical scattering of the individual values. Therefore, there can be no doubt that a significant, progressive decrease occurred. On the other hand, no trend can be discerned in the values of the water content in the soil at the center of a

surrounding group of piles (Fig. 9(*b*)). For all practical purposes the water content at the middle of the group remained unchanged throughout the 11 months that observations were made.

Interpretation of Results

The water content data demonstrate that a process of consolidation was initiated by pile driving. The water content adjacent to the piles decreased unmistakably with the passage of time, implying that the water must have migrated away from the disturbed region. It could not have gone into the piles, because the piles consisted of watertight steel shells with joints 12 ft apart in the vertical direction. The joints themselves were waterproofed. However, even if the joints had been able to serve as drains, the vertical spacing between joints was much greater than the horizontal spacing between piles, and considerable time would have been required for water to flow vertically to find any possible openings. It is also apparent that the water did not flow upward through the clay mass to the laminated clay where drainage surfaces might have existed in the form of thin pervious layers. Fig. 9 shows a practically constant decrease in water content from top to bottom of the 20-ft layer from which the samples were taken, thus virtually eliminating the possibility that the water migrated in a vertical direction.

The only alternative appears to be that the water migrated horizontally from the disturbed zone near the pile to the much larger mass of relatively undisturbed clay between the piles. A very slight swelling of the more undisturbed clay, corresponding to an imperceptible increase in water content, would be sufficient to permit an appreciable drop in the water content in the relatively small volume of disturbed clay surrounding the pile.

Since the water appears to have migrated exclusively in a horizontal direction, the process of consolidation initiated by driving friction piles into the clay would not ordinarily result in settlement. Furthermore, as the consolidation is accompanied by a swelling in other parts of the clay mass, no net volume change would occur and there would be no reason to expect the ground surface to subside.

Conclusions

The preceding information indicates that the driving of piles at the site where the investigation was made did not have the detrimental effects which were forecast on the basis of soil tests on undisturbed and remolded samples and the theory of consolidation as applied by Professor Casagrande. The soil adjacent to the pile was not completely remolded but, on the contrary, was disturbed only to an extent that, at the most, might be called "half remolded." For all practical purposes the soil in the middle of the pile group was not remolded at all.

It is also evident that the type of consolidation produced by the pile driving did not involve a vertical flow of water out of the soil, but only a lateral flow. Therefore, if the bridge foundation had been supported by friction piles rather than point bearing piles, there is no reason to believe that the consolidation would have produced a settlement of the ground surface. The assumption

that the settlement would proceed as a result of vertical migration of the water out of the clay, which formed the basis of the Casagrande settlement forecast, is not in agreement with the observations made in the investigation. Therefore, it is extremely doubtful that the disturbing effect of pile driving, even for friction foundations, would be detrimental in clay of the type encountered. On the contrary, it is quite probable that piles would be beneficial.

The clays at the site of the investigations varied considerably in sensitivity to disturbance, the stratified clays being much more sensitive than the unstratified clays. However, the evidence in the investigation does not warrant the conclusion that disturbance due to driving piles into soft clay is a source of detrimental decrease in the strength of these clays or a source of excessive settlement.

In the first place, the region of disturbance is limited to a zone within a few inches of the pile and the main volume of the clay mass is apparently undisturbed. In the second place, based on the unconfined compressive strength, the character of the changes in soil structure were apparently such as to increase the strength in the unstratified clay and a large portion of the decrease in the stratified clay was recovered with time. In the third place, the change in moisture content in the disturbed zone left no evidence of migration of moisture from the clay mass as a whole. Under these conditions it is difficult to see how any settlement of the surface could result.

Acknowledgments

This investigation was a cooperative enterprise of the Raymond Concrete Pile Company (New York, N. Y.), the Michigan State Highway Testing Laboratory, University of Michigan at Ann Arbor, and the University of Illinois (Urbana). The authors are indebted to their colleagues in the several organizations for valuable assistance in the field and laboratory work and in the subsequent analysis of the test data.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

CITIES CANNOT LIVE WITHOUT TRUCKS

BY HOY STEVENS [1]

All goods or commodities purchased by the American people for personal use travel by motor truck at some stage, and usually during many stages, of their journey between the original raw material producer and the ultimate consumer. Any apparent exceptions are goods, materials, or machinery for the use of intermediate industries that either fabricate or transport consumer goods.

As a reminder of the way in which trucks have been intimately woven into the economic pattern, some of the uses of trucks in cities include:

1. Delivery of goods and commodities (*a*) from retailers to ultimate consumers; (*b*) from warehouses to wholesalers, to retail outlets, or to other users; (*c*) from producers and manufacturers to wholesalers or retailers; and (*d*) to and from the collection depots of long distance carriers (such as highway motor truck terminals, railroad terminals, air cargo carrier airports, and steamship docks).

2. Transportation of the service employees of utilities and other service industries that help to make our cities livable and efficient—for example, the Bell Telephone System uses more than 18,000 trucks to maintain its service.

CHICAGO (ILL.) CASE HISTORY

In order to give a more specific idea of the importance of motor trucks to a city, some of the data from a report[2] on a comprehensive study made in 1945 of the flow of truck traffic in and out of Chicago are presented. Data from this study and report, prepared by the Illinois Highway Traffic Advisory Committee to the War Department, indicate the huge volume of goods which must be handled daily over the streets of Chicago in order to meet the needs of the community. It states in part:

> "The role played by highway transportation in war production was only an extension of its service to the pre-war industrial economy, a service which without a doubt will be further expanded in post-war years. The trucking industry today employs about 4,000,000 people in the United States. Scarcely an item of commerce is produced which is not dependent at some point on motor transportation. Trucks traveling on highways are the sole means of freight delivery to over 54,000 small communities."

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1949.**

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[2] "Chicago Truck Terminal—1945," Illinois Highway Traffic Advisory Committee to the War Dept. (lithograph).

Table 1, taken from the Chicago report, shows the tons of freight carried daily into and out of the Chicago area by various types of vehicles and by different types of operators. In all, 32,000 truck units carry 80,774 tons of freight into or out of the area daily. The report states:

> "About ten percent of these (5,000 single unit) carrier-operated trucks have an origin or destination at a transportation terminal. From data of origin-destination, these trucks appear to be engaged in the complicated task of distributing the necessities of life throughout the primary trade areas, and in transporting the goods of the region between numerous warehouses, wholesale stores, retail stores, etc. The over-the-road motor carrier appears to use the single unit trucks largely for the local distribution of goods.
>
> "Of the 6,300 combination units operated daily into and out of Chicago, approximately 1,800 have a definite origin or destination or both in a transportation terminal, and some 1,800 more have an origin or destination or both at a warehouse, which certainly would include characteristics of a freight terminal. * * * These vehicles operate into rail-truck interchange terminals, motor-transport terminals, and, to a minor extent, into air terminals. It is not possible to estimate the tonnage of freight delivered to these various types of terminals from the traffic data alone; but it is estimated that, of the 50,000 tons of freight handled daily into and out of Chicago, at least two fifths, or 20,000 tons, is destined for motor transportation terminals for local delivery, or for interchange with other motor carrier lines.
>
> "Of the 50,000 tons of freight being dispatched daily, about 30,000 tons (estimated by various trucks at 55% to 70%) is handled directly from the shipper's or to the consignee's docks in full truck loads, and without having been first unloaded on a terminal dock for distribution."

TABLE 1.—DAILY TRUCK OPERATION INTO AND OUT OF THE CHICAGO (ILL). AREA

Type of vehicle	Commercial Motor Carriers		Privately Owned Vehicles	
	No.	Tonnage	No.	Tonnage
(1)	(2)	(3)	(4)	(5)
Trucks	5,000	9,471	18,800	22,753
Tractor-trailer trains	6,300	40,640	1,900	7,910
Total	11,300	50,111	20,700	30,663

This information not only shows the essential daily truck needs of a city but also indicates the enormous traffic problem which such vehicular flow presents.

FALLACIES IN THINKING ABOUT TRUCKS

Before considering possible remedies for congested street conditions, it would be well to analyze critically some of the fallacies that have crept into popular thinking regarding commercial motor vehicles. Too frequently "sidewalk" thinking about traffic cures overlooks two basic facts: (*a*) That large American communities could not exist without modern truck transportation, and (*b*) that the volume of truck freight is so large and so important that it must be moved continuously and during the regular working hours of business establishments.

One comment frequently heard is "great big trucks." Actually, in so far as frontal width is concerned, none of the regular goods-carrying trucks are larger

than other vehicles which are accepted on the highways; namely, vehicles used in the public transportation of passengers. Large straight trucks are no bigger than large buses or trolley coaches, whereas tractor-trailer trains are comparable to streetcars. Admittedly, van-type trucks frequently are higher than other vehicles on the road, generally from 12 ft to 12½ ft, but the height of the vehicle has nothing to do with the area it occupies as it travels through the streets.

The trucking industry prefers to use large units because they are most economical and most efficient. In the last analysis, the customer prefers the most efficient vehicles. However, it may be desirable to define small trucks as to size. It will no doubt be agreed that it will require at least 2 or 3 small van trucks to take the place of 1 large van trailer. Certainly, substituting 2 or 3 small vehicles for 1 large vehicle will not improve traffic situations. The Chicago data estimates that 30,000 tons daily were handled directly from shippers' docks or to consignees' docks in full truck loads without having first been unloaded on the carriers' docks for distribution in small trucks. In thinking of remedial measures, it must not be assumed that the use of 2 or 3 city delivery trucks in place of 1 large efficient van is a remedy for traffic congestion.

Survey of Congestion Complaints

The American Transit Association has made available some data regarding the interference of trucks with the buses, trolley coaches, and streetcars of urban public transportation companies. This information is thought to be particularly valid as an indication of conditions on city streets because the public transportation companies have regularly scheduled operations over the city streets and are thereby more quantitatively able to judge actual interference. Two of the questions asked of the local transportation companies in a questionnaire circulated on June 21, 1946, were:

1. Are your vehicles being delayed and your schedules disrupted by trucks loading and unloading on the streets? and
2. How serious is the situation caused by the trucks; that is, how would you rate vehicle blocks due to trucks among the factors responsible for delays to your vehicles?

A recapitulation of the replies from the various companies shows the answers to the first question to be: Yes, 21; slightly, 14; and no, 14. Answers to the second question were: Minor problem, 29; serious problem, 10; coal trucks, 2; and double parking, 5.

The purpose of presenting these data is merely to emphasize the point that in so far as these regular and mature users of the streets are concerned, trucks are a part of the traffic problem, but definitely are far from the major cause of traffic delays.

Certain Unsatisfactory Expedients

From New York, N. Y., which has almost reached the ultimate of traffic loads with little possibility of doing anything but keeping even with its problems come the following comments (from the special inquiry service of the American Transit Association), which are thought to be significant:

"Various experiments in traffic regulations have been made. As an example, effective in October 1945, the police department prohibited parking along 34th Street between Second and Ninth Avenues from 7:00 A.M. to 7:00 P.M. daily except Sundays. Prior to this prohibition some week-day service on the 34th Street route was considerably behind schedule. Within a few days thereafter, schedules were fairly maintained. However, in the months following the parking restrictions, the volume of moving vehicular traffic along 34th Street has increased to such an extent that the benefits accruing from the parking regulations have largely disappeared. Coach service along 34th Street is again operating behind schedule."

Contained also in these comments are some replies from cities where other expedients had been tried. For example, night deliveries were suggested, but labor organizations oppose them because their members do not want to work nights. Businessmen oppose night deliveries because they would have to maintain a night force and because their establishments would be more vulnerable to robbery and theft when opened up for night delivery.

The Problem in Regard to Trucks

There is no doubt that trucks at rest while being loaded and unloaded are a problem. How can this loading and unloading situation be remedied? In so far as it can be seen it is a matter of off-street loading areas. This idea is not new, but this seems to be the most feasible answer. The common carriers have to load and unload at their shippers' and receivers' facilities. The private truck operator has some choice of his own loading space facilities, especially if he owns the plant or building. Tenants of buildings may have less choice. Fundamentally, the problem of standing trucks for loading or unloading is a matter of building design and all truck operators will welcome any efforts to convince plant and building owners that it is sound business for them to provide ample off-street truck loading areas.

The earliest ordinance requiring off-street loading facilities for industrial or commercial buildings dates back to June, 1931, in Pueblo, Colo.[3] In 1946, zoning ordinances of twenty-one cities required retail stores and wholesale houses to furnish adequate off-street loading facilities. Twenty cities require industrial buildings to furnish off-street loading facilities. Seventeen cities have this requirement for office buildings.

No city has passed a retroactive ordinance which would compel the installation of off-street loading facilities in buildings already in existence at the time the ordinance was passed. However, one city (Greenville, S. C.) has set a period of eighteen years from the date of the ordinance during which the building may continue to exist without conforming to the ordinance. A number of cities now provide that, if a building is substantially altered or repaired, the off-street parking or loading provisions must be complied with.

Different thinking is needed as to what can or may be done in regard to off-street loading. Why is it necessary to continue to think that alternate routes, overhead routes, and by-passes must be built to permit industrial areas to continue in their motley array of curbstone loadings? Consider what the result

[3] "Zoning Applied to Parking," by C. S. McGraw, Jr., and Wilbur S. Smith, *Traffic Quarterly*, The Eno Foundation for Highway Traffic Control, Inc., Saugatuck, Conn., January, 1947.

would be if overhead roadways were built over commercial streets. Actually, off-street truck loading areas would be provided at the curbstones of the lower street to the extent of each building's front footage. This arrangement still might not provide adequate loading areas for certain buildings and might provide too much for other buildings. In addition, the overhead road structures would depress the value of the buildings to the detriment of their owners, and depress the tax values of the buildings to the detriment of the community.

Amount of Off-Street Loading Area

The several ordinances regarding off-street loading facilities are based on three general premises. Some prescribe loading spots based on the gross floor area in the building, others prescribe certain minimum space between an alley and the rear of the building, and others merely prescribe reasonable or adequate facilities so as not to interfere with traffic. Obviously, before any sound development of off-street loading can be had there is a need for formulas by which reasonable or adequate facilities can be estimated.

The New York Regional Plan Association[4] gives a sliding scale for use in computing the required number of truck berths, the number of loading areas being based on the gross floor area for two types of building—office buildings and hotels and buildings for manufacturing, storage, and similar facilities. It may be that loading area formulas based on floor area are types which can produce reasonably consistent results. However, any formula based on floor area overlooks one fundamental—namely, that the trucks move goods and not floor area. Truck loading area requirements should be based on the volume and tonnage of freight to be moved into and out of a building.

The American Trucking Association is engaged in a study of loading platform areas with reference to the tonnage of freight handled. This study, also, is considering the areas required for efficient manipulation of vehicles in front of the loading area. The material is not complete, but some estimated values will indicate the type of coefficients that may be developed. When available the results of this study should be of real assistance in city planning.

For rapid and efficient manipulation of commercial vehicles when they are to be backed against a dock for loading or unloading, the open area between the edge of the loading platform and the outer edge of the manipulation area should be approximately twice the length of the vehicle. This manipulation area may be wholly within the building, or the outer half may be a part of an alley or a street. On heavily traveled streets all the area obviously should be within the property lines. Along the edge of the dock from 10 ft to 12 ft should be allowed per vehicle. These dimensions will permit expeditious backing of trucks into loading stalls.

Of major importance in speeding the loading and unloading of commercial vehicles is the size of the dock area adjoining the trucks. To some extent, the dock area necessarily will be dependent on material handling equipment and arrangements for transporting the freight between the loading dock and the more permanent storage area of the commodities being received or shipped.

[4] "Parking and Traffic Study," New York Regional Plan Assn., New York, N. Y., 1942, p. 42.

Sufficient area should be provided on the dock for the truck load accumulation of outgoing shipments or for the truck load receipt of incoming shipments without the necessity of moving the freight to or from the dock while the truck is held at the dock.

Based on the experience of the trucking industry, it is necessary that the dock area be at least twice the total body floor area of the largest number of trucks that can be docked at one time. This is a logical conclusion when it is realized that freight of average density (from 22 lb to 24 lb per cu ft) usually is piled between 5 ft and 6 ft high in the truck, is transported to and from the truck on two-wheeled hand trucks handling a height of stock from 3 ft to 4 ft, and seldom is tiered on the loading dock; hence, allowing for aisles in which the hand trucks can operate, the required dock area is at least twice the floor area of the vehicles at the dock.

With such a dock area it would be possible to accumulate an entire outgoing load for a vehicle—and in a receiving section to have enough area upon which to unload an entire vehicle—opposite each loading spot. Even with dock areas of this size ratio it will still be necessary to transport the freight promptly to and from the dock and building in order to make continuous use of the truck berths. If a loading dock is to be used as a storage area even more space is required. Final formulas will be more specific than this, but this value indicates the basis of present thinking.

In the case of loft buildings, elevator service for the upper floors will be a great factor in determining dock area. Freight placed on an elevator is seldom tiered; hence, calculations of elevator size may be made on an area basis to determine elevator requirements.

Conclusions

Because cities cannot live without trucks, it is the duty of the communities to plan for or to provide facilities that will reduce the truck factor in traffic congestion for the benefit of all vehicular movement, including trucks. The trucking industry is definitely interested in remedies that will reduce traffic congestion and will improve facilities at shipping and receiving establishments for loading and unloading motor trucks. The business of that industry is moving goods and commodities, and trucks make no money when delayed in traffic or when hindered in loading or unloading.

Individually, as well as an industry, truck operators are becoming more and more interested in problems of traffic as well as in problems of loading and unloading trucks at curbs. The state trucking associations and their members are cooperating in local studies of traffic, zoning, and freight handling facilities in cities where street traffic conditions are unsatisfactory. It is believed that the truck operators have developed sound experience in regard to the handling of general freight. The dock facilities that they have found efficient should give adequate data for shippers and receivers. When the data are complete, it is planned to pass on this accumulated experience in the form of specific recommendations regarding loading docks for use with motor freight vehicles.

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PAPERS

DIFFUSION OF SUBMERGED JETS

BY M. L ALBERTSON,[1] JUN. ASCE, Y. B. DAI,[2] R. A. JENSEN,[3] AND HUNTER ROUSE,[4] M. ASCE

SYNOPSIS

As the direct result of turbulence generated at the borders of a submerged jet, the fluid within the jet will undergo both lateral diffusion and deceleration, and at the same time fluid from the surrounding region will be brought into motion. The approximate characteristics of the corresponding mean flow pattern are derived analytically, with the exception of a single experimental constant, through assumptions that: (1) The pressure is hydrostatically distributed throughout the flow; (2) the diffusion process is dynamically similar under all conditions; and (3) the longitudinal component of velocity within the diffusion region varies according to the normal probability function at each cross section. Experimental data are presented which justify the analysis and provide the necessary coefficients for flow from both slots and orifices. All results are reduced to a form immediately useful for design purposes.

STATEMENT OF THE PROBLEM

As far as most engineers are concerned, high-velocity flow from a submerged outlet represents merely an irrecoverable loss of power, for a basic axiom of hydraulics states that the entire kinetic energy of such a jet will be dissipated through reaction with the surrounding fluid. In more explicit terms, the difference in velocity between a jet and the region into which it is discharged will give rise to a pronounced degree of instability, the kinetic energy of the oncoming flow steadily being converted into kinetic energy of turbulence, and the latter steadily decaying through viscous shear. However, such conversion is by no means restricted to the vicinity of the outlet, for any reduction in kinetic energy necessarily represents a decrease in the velocity of flow, and even the

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1949.**

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most elementary considerations of continuity indicate that the area of the flow section must become great as the velocity becomes small. In view of the Newtonian principle of action and reaction, moreover, it will be realized that deceleration of the fluid in the jet can occur only through simultaneous acceleration of the surrounding fluid, so that the total rate of flow past successive sections of the jet will actually increase with distance from the outlet.

The phenomenon of "exit loss" so simply treated in hydraulics evidently involves far more than a local energy conversion. Indeed, a detailed knowledge of the velocity distribution within such an expanding stream is essential to many problems of diffusion. Flow of a liquid suspension from a submerged pipe into a settling basin is a case in point, although a closely related phase of mixing is found in the efflux of heated or cooled air from a ventilation duct in the wall of a room; in either case the engineer is interested in the shape and the "carry" of the jet, and in the amount of entrainment by (and hence dilution of)

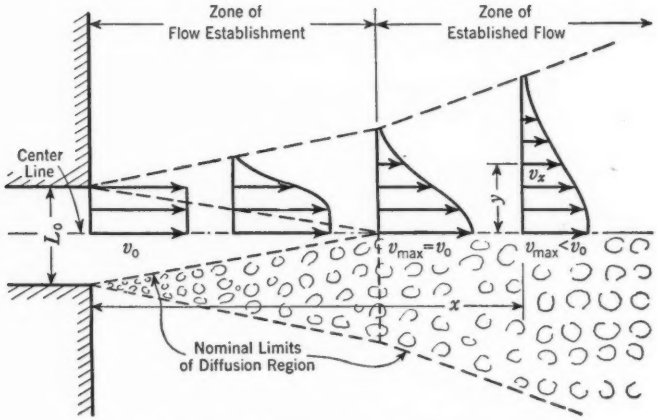


FIG. 1.—SCHEMATIC REPRESENTATION OF JET DIFFUSION

the jet from section to section. Directly related, although more closely restricted by boundary (that is, free-surface) conditions, is the diffusion of plunging nappes from spillways or of submerged jets from sluice gates, because information as to the accompanying rate of deceleration is often necessary in the prevention of scour beyond the apron. Finally, since the slip stream of a propeller differs little from any other type of jet, the velocity distribution in the wake of aircraft and watercraft (a matter of vital importance, for example, in submarine detection) should be subject to the same method of analysis.

With reference to Fig. 1, it will be seen that an initial zone of flow establishment must exist beyond the efflux section of either a two-dimensional or a three-dimensional submerged jet. Since the fluid discharged from the boundary opening may be assumed to be of relatively constant velocity, at the efflux section there will necessarily be a pronounced velocity discontinuity between the jet and the surrounding fluid. The eddies generated in this region of high shear will immediately result in a lateral mixing process which progresses both

inward and outward with distance from the efflux section. Such lateral mixing produces a necessarily balanced action and reaction: On the one hand, the fluid within the jet is gradually decelerated; on the other hand, fluid from the surrounding region is gradually accelerated or entrained. As a result, the constant-velocity core of the jet will steadily decrease in lateral extent, whereas both the rate of flow and the over-all breadth of the jet will steadily increase in magnitude, with distance from the efflux section. The limit of this initial zone of flow establishment is reached when the mixing region has penetrated to the center line of the jet.

Once the entire central part of the jet has become turbulent, the flow may be considered as fully established, for the diffusion process continues thereafter without essential change in character. Further entrainment of the surrounding fluid by the expanding eddy region is now balanced inertially by a continuous reduction in the velocity of the entire central region. Such variation approaches (but never actually attains) a limit as the center-line velocity becomes of negligible magnitude a very great distance from the original efflux section, the lateral extent of the eddy region—and hence the quantity of fluid within the jet—then being wholly out of proportion to conditions at the initial section.

Just as the theoretical limit of the zone of established flow is evidently nonexistent, the border section between this zone and the zone of establishment (indicated by the vertical broken line in Fig. 1) can be considered sharply defined only to the extent to which one may neglect the zone of transition which necessarily exists between two distinct flow regimes. There is, in fact, no precise point at which the eddies from opposite sides of the jet can be said to meet, for the statistical nature of the mixing process makes it impossible to place more than an arbitrary limit on its lateral range. The boundaries of the diffusion region, like the border between the two zones, must therefore be accepted merely as convenient nominal designations.

Conditions within the zone of flow establishment and the zone of established flow were first investigated theoretically by W. Tollmien in 1926,[5] on the assumptions that: (*a*) The sole effective force was the tangential shear expressed in terms of the lateral momentum transport of the mixing process; (*b*) the mixing length varied with the first power of the longitudinal distance from the efflux section; and (*c*) the velocity of the turbulence was proportional to the product of the mixing length and the mean velocity gradient. With the exception of a numerical coefficient which had to be evaluated experimentally, Mr. Tollmien was able to derive a stream function which indicated with good approximation the actual pattern of the mean flow in each region.[6] Later studies by other investigators[7,8,9] introduced either the concept of vorticity transport or the assumption of constant eddy viscosity across any section of the diffusion zone,

[5] "Berechnung turbulenter Ausbreitungsvorgänge," by W. Tollmien, *Zeitschrift für angewandte Mathematik und Mechanik*, Vol. 6, 1926, p. 468.

[6] "Turbulente Ausbreitungsvorgänge im Freistrahl," by P. Ruden, *Die Naturwissenschaften*, Vol. 21, 1933, p. 375.

[7] "Investigations of the Turbulent Mixing Regions Formed by Jets," by A. M. Keuthe, *Journal of Applied Mechanics*, September, 1935, p. A-87.

[8] "Application of the Modified Vorticity Transport Theory to the Turbulent Spreading of a Jet of Air," by S. Tomotika, *Proceedings*, Royal Soc. of London, Vol. 165, Series A, March–April, 1938, p. 65.

[9] "Concerning the Velocity and Temperature Distribution in Plain and Axially Symmetrical Jets," by L. Howarth, *Proceedings*, Cambridge Philosophical Soc., Vol. 34, 1938, p. 185.

but did not add materially to the agreement with experimental measurements. As a matter of fact, the sensitivity of the mean velocity-distribution function to the type of turbulence structure assumed is not great; the comparison between measurement and theory has, as a result, never been sufficient to provide a conclusive check on the accuracy of the assumed mixing characteristics. Indeed, not until detailed measurements of the turbulence within two-dimensional and three-dimensional jets were made by S. Corssin[10] and by H. W. Liepmann and J. Laufer[11] was the considerable discrepancy between assumption and fact fully realized.

Since an approximate analysis of the mean velocity distribution within either zone actually requires no assumption as to the distribution of turbulence, and since the characteristics of jet diffusion have not yet been published in a form readily usable by engineers, this paper seeks to fulfil this twofold purpose. The dimensional aspects of the phenomenon are first discussed, followed by an elementary physical analysis of the mean flow pattern. Experimental data covering a considerable range of each independent variable for both two-dimensional and three-dimensional flow are then presented to justify the analysis and to provide the single numerical coefficient still required for each case. All results are finally summarized in a convenient dimensionless form.

Analysis of the Mean Flow Pattern

General Considerations.—If the Reynolds number for fluid efflux from a submerged boundary outlet is not too low, the mean velocity v at any point (see Fig. 1) should depend only on the coordinates x, y, and z, on the efflux velocity v_0, and on a linear dimension L_0 characterizing the particular outlet form. These variables may be grouped in the dimensionless relationship:

$$\frac{v}{v_0} = f_1\left(\frac{x}{L_0}, \frac{y}{x}, \frac{z}{x}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This relationship must be considered to involve the magnitude and the direction of the vector v, the components of which may further be related through the differential equation of continuity:

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The rate of flow or volume flux Q past successive normal sections may be written as the integral of the differential flux $v_x\,dA$ over any normal section. Since, because of entrainment, Q will vary with the longitudinal distance x from the efflux section, its ratio to the efflux rate Q_0 may be expected to have the functional form:

$$\frac{Q}{Q_0} = \frac{\int_0^\infty v_x\,dA}{v_0 A_0} = f_2\left(\frac{x}{L_0}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

[10] "Investigation of Flow in an Axially Symmetrical Heated Jet of Air," by S. Corssin, *Advance Confidential Report No. 3L23*, National Advisory Committee for Aeronautics, Washington, D. C., 1943.

[11] Investigations of Free Turbulent Mixing," by H. W. Liepmann and J. Laufer, *Technical Note No. 1257*, National Advisory Committee for Aeronautics, Washington, D. C., August, 1947.

in which A_0 is the cross-sectional area of the outlet. Similarly, since the momentum flux M may be written as the integral of the volume flux $v_x\,dA$ times the longitudinal component of momentum per unit volume $\rho\,v_x$, ρ being the fluid density, the ratio of M for any section to M_0 for the efflux section should be

$$\frac{M}{M_0} = \frac{\int_0^\infty (v_x)^2\,dA}{(v_0)^2\,A_0} = f_3\left(\frac{x}{L_0}\right) \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Finally, since the energy flux E is expressible as the integral of the volume flux $v_x\,dA$ times the kinetic energy per unit volume $\rho\,v^2/2$, the ratio of E for any section to E_0 for the boundary outlet should take the form:

$$\frac{E}{E_0} = \frac{\int_0^\infty v^2\,v_x\,dA}{(v_0)^3\,A_0} = f_4\left(\frac{x}{L_0}\right) \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Such general considerations will yield no further clue to the actual functional relationships without specific statements as to the dynamics of the flow. For example, on the basis of experimental evidence it may safely be presumed that the pressure distribution is essentially hydrostatic throughout the zone of motion—in other words, that the sole force producing the deceleration of the jet and the acceleration of the surrounding fluid is the tangential shear within the mixing region. Because this process is wholly internal, it follows at once that the momentum flux must be a constant for all normal sections of a given flow pattern:

$$\frac{M}{M_0} = \frac{\int_0^\infty (v_x)^2\,dA}{(v_0)^2\,A_0} = 1 \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

If, moreover, viscous action is presumed to have no influence on the mixing process, the diffusion characteristics—and hence the characteristics of the mean flow—should be dynamically similar under all conditions. Thus, in effect, the same velocity function must characterize every section within the diffusion region. As a matter of fact, experimental data follow the general trend of the Gaussian normal probability function:

$$\frac{v_x}{v_{max}} = \exp\left(-\frac{y^2}{2\,\sigma^2}\right) \ldots\ldots\ldots (7)$$

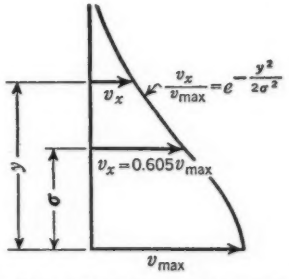


FIG. 2.—CHARACTERISTICS OF THE NORMAL-PROBABILITY CURVE

in which the right-hand member expresses the value e^n, the quantity in parentheses being the value of n. (A similar simplification applies subsequently in Eqs. 12, 19, 24, and 29.) The use of Eq. 7 in the analysis permits the characteristics of the entire flow pattern to be expressed in terms of the two parameters which define the proportions of the curve: the velocity v_{max} and the standard or root-mean-

square deviation σ (see Fig. 2). Moreover, not only does Eq. 1 then reduce to

$$\frac{v_{max}}{v_0} = f_5\left(\frac{x}{L_0}, \frac{\sigma}{x}\right) \quad \ldots\ldots (8)$$

but the condition of dynamic similarity simultaneously requires that at all cross sections, regardless of the efflux velocity,

$$\frac{\sigma}{x} = C \quad \ldots\ldots (9)$$

In other words, the angle of jet diffusion must be constant.

Through use of Eqs. 6, 7, and 9, all characteristics of the mean flow pattern for any specific boundary condition may now be determined analytically, with the exception of a single experimental coefficient. Such characteristics include the variation in the magnitude and direction of the velocity vector with coordinate space, and the variation of volume flux and energy flux with distance from the outlet section. In the following pages these characteristics are evaluated for the zone of flow establishment and for the zone of established flow under both two-dimensional and three-dimensional conditions.

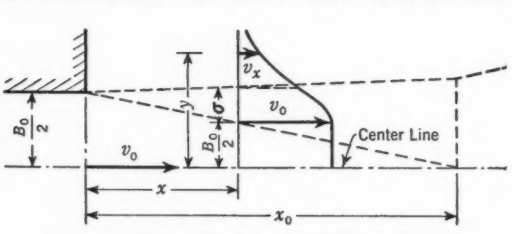


FIG. 3.—DEFINITION SKETCH FOR ZONE OF FLOW ESTABLISHMENT

Zone of Flow Establishment.—In the case of efflux from a long slot, it will be noted from Fig. 3 that v_{max} will correspond to v_0 as long as the eddies have not penetrated to the central plane of the jet. Therefore, the distribution of v_x across any normal section within this zone may be represented as a first approximation by the two symmetrical halves of the probability curve connected with a straight line through the constant-velocity core. Evaluation of the momentum-flux integral of Eq. 6, for the condition that $\sigma/x = C_1$, will be found to yield at once the specific relationship:

$$\frac{x_0}{B_0} = \frac{1}{\sqrt{\pi}\, C_1} \quad \ldots\ldots (10)$$

in which x_0 represents the distance from the efflux section to the end of the zone of establishment. Evidently, the nominal inner boundary of the diffusion region is a plane satisfying the expression:

$$\frac{B}{B_0} = 1 - \frac{x}{x_0} \quad \ldots\ldots (11)$$

The distribution function for v_x in the diffusion region then takes the explicit form:

$$\frac{v_x}{v_0} = \exp\left[-\frac{\left(y + \sqrt{\pi}\, C_1 \frac{x}{2} - \frac{B_0}{2}\right)^2}{2\,(C_1 x)^2}\right] \quad \ldots\ldots (12)$$

Similar evaluation of the volume-flux integral of Eq. 3, after introduction of v_x from Eq. 12, will result in the relationship:

$$\frac{Q}{Q_0} = 1 + \sqrt{\pi}\,(\sqrt{2} - 1)C_1 \frac{x}{B_0} \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Inasmuch as the rate of change in volume flux with longitudinal distance dQ/dx must equal twice the velocity of entrainment (that is, the velocity v_y at a considerable distance y from the jet), it follows that

$$\operatorname*{Limit}_{y\to\infty} \frac{v_y}{v_0} = -\tfrac{1}{2}\sqrt{\pi}\,(\sqrt{2} - 1)C_1 \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

The same result would be obtained as a limit through solution of the differential equation of continuity for v_y as a function of x and y; however, since the latter function must be shown graphically rather than algebraically, it is not introduced at this point. (All detailed evaluations of this nature have been deposited with the Engineering Societies Library[11a] for reference.)

Although the energy-flux integral properly involves the magnitude of the velocity vector in the term for kinetic energy per unit volume, the vector magnitude may be expected to differ little from that of its longitudinal component except in the outer regions, and the contribution of these regions to the total flux should be relatively small. The energy-flux ratio may therefore be approximated in terms of v_x, as follows:

$$\frac{E}{E_0} = 1 + \sqrt{\pi}\,(\sqrt{\tfrac{2}{3}} - 1)C_1 \frac{x}{B_0} \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

Flow from a circular orifice should differ from the foregoing only in the substitution of the orifice diameter D_0 for the slot width B_0 and the substitution of the radial distance r for the lateral distance y in the basic equations, the integration being performed over the corresponding circular areas for the condition that $\sigma/x = C_2$. At the limit $x = x_0$, the momentum relationship at once results in

$$\frac{x_0}{D_0} = \frac{1}{2\,C_2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

which is a counterpart of Eq. 10. However, the general solution of the momentum relationship yields an expression which—in contradistinction to Eq. 11 for the two-dimensional case—indicates that the diameter of the nominal inner border of the diffusion region for three-dimensional flow is not a linear function of x; that is,

$$\frac{D}{D_0} = \sqrt{1 + (\pi - 4)(C_2)^2 \frac{x^2}{(D_0)^2}} - \sqrt{\pi}\,C_2 \frac{x}{D_0} \ldots\ldots\ldots\ldots (17)$$

Fortunately, the difference between this curvilinear function and the linear approximation

$$\frac{D}{D_0} = 1 - \frac{x}{x_0} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

[11a] 29 West 39th St., New York 18, N. Y.

is probably no greater than the discrepancy between the actual velocity distribution and the assumed probability curve. For all practical purposes it is sufficient, therefore, to write the velocity distribution within the region of diffusion in the simplified form:

$$\frac{v_x}{v_0} = \exp\left[- \frac{(r + C_2 x - D_0/2)^2}{2 (C_1 x)^2} \right] \quad \ldots\ldots\ldots\ldots\ldots (19)$$

In terms of this simplification, the equation for the variation in the volume-flux ratio with distance from the orifice then becomes

$$\frac{Q}{Q_0} = 1 + 2 (\sqrt{2\pi} - 2) C_2 \frac{x}{D_0} + 4 (3 - \sqrt{2\pi}) (C_2)^2 \frac{x^2}{(D_0)^2} \ldots\ldots (20)$$

Although, as in the two-dimensional case, the rate of entrainment may now be expressed as the derivative of the volume flux with longitudinal distance, for reasons of continuity the radial velocity v_r beyond the diffusion region will vary inversely with radial distance from the center line; in other words, the product $r\, v_r$ rather than v_r itself approaches a finite limit in such three-dimensional flow:

$$\underset{r \to \infty}{\text{Limit}} \frac{v_r}{v_0} \frac{r}{D_0} = - \tfrac{1}{4} (\sqrt{2\pi} - 2) C_2 - (3 - \sqrt{2\pi}) C_2 \frac{x}{D_0} \ldots\ldots (21)$$

The variation of v_r within the diffusion zone may be evaluated from the differential equation of continuity; since this function must again be shown graphically rather than algebraically, it is not presented herein. Finally, as in the

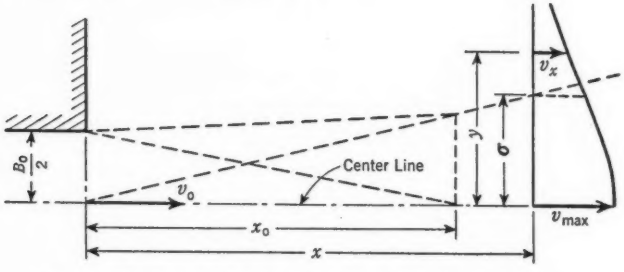


FIG. 4.—DEFINITION SKETCH FOR ZONE OF ESTABLISHED FLOW

case of flow from a slot, it may be assumed that the difference between the magnitude of the velocity vector and that of its longitudinal component will have little effect on the expression for the energy flux past successive sections beyond an orifice, which thus becomes

$$\frac{E}{E_0} = 1 + 2 \left(\sqrt{\frac{2\pi}{3}} - 2 \right) C_2 \frac{x}{D_0} + 4 \left(\frac{5}{3} - \sqrt{\frac{2\pi}{3}} \right) (C_2)^2 \frac{x^2}{(D_0)^2} \ldots (22)$$

Zone of Established Flow.—As will be noted from Fig. 4, at all sections beyond the zone of flow establishment the probability curve indicating the velocity distribution will be continuous across the line of symmetry, and the ratio σ/x will therefore become directly indicative of the rate of outward spread

of the eddy region. Assuming, for the two-dimensional case, that $\sigma/x = C_1$, evaluation of the momentum flux will lead at once to the expression:

$$\frac{v_{max}}{v_0} = \sqrt{\frac{1}{\sqrt{\pi}\, C_1}\frac{B_0}{x}} \qquad (23)$$

in which the quantity $B_0/(\sqrt{\pi}\, C_1)$ represents the same parametric distance x_0 as that in Eq. 10. In other words, determination of the single constant C_1 from experimental measurement will suffice for both zones. The distribution of the longitudinal velocity component in the zone of established flow may then be written as

$$\frac{v_x}{v_0} = \sqrt{\frac{1}{\sqrt{\pi}\, C_1}\frac{B_0}{x}}\, \exp\left[-\frac{1}{2(C_1)^2}\frac{y^2}{x^2} \right] \qquad (24)$$

From the equation for volume flux it will be found, for the established zone, that

$$\frac{Q}{Q_0} = \sqrt{2\sqrt{\pi}\, C_1 \frac{x}{B_0}} \qquad (25)$$

and differentiation with respect to x will yield the rate of entrainment and the corresponding lateral velocity at a considerable distance from the jet:

$$\underset{y\to\infty}{\text{Limit}}\, \frac{v_y}{v_0} = -\sqrt{\frac{\sqrt{\pi}\, C_1}{8}\frac{B_0}{x}} \qquad (26)$$

Eq. 26 will also be the limit of the general function for v_y obtained by graphical solution of the equation of continuity and reproduced at a later point. Evaluation of the energy flux in terms of $v_x \approx v$ finally leads to the approximate expression:

$$\frac{E}{E_0} = \sqrt{\frac{2}{3\sqrt{\pi}\, C_1}\frac{B_0}{x}} \qquad (27)$$

Once more replacing y and B_0 by r and D_0, and integrating over the corresponding areas, the equivalent expressions for the three-dimensional case may be evaluated for the condition that $\sigma/x = C_2$. The momentum equation at once reduces to

$$\frac{v_{max}}{v_0} = \frac{1}{2\, C_2}\frac{D_0}{x} \qquad (28)$$

in which the quantity $D_0/(2\, C_2)$ corresponds to the same parametric distance x_0 as that in Eq. 16. Evidently, determination of the single experimental constant C_2 will suffice to define all characteristics of both zones, just as in the two-dimensional case. The remaining expressions then become

$$\frac{v_x}{v_0} = \frac{1}{2\, C_2}\frac{D_0}{x}\, \exp\left[-\frac{1}{2\,(C_2)^2}\frac{r^2}{x^2} \right] \qquad (29)$$

$$\frac{Q}{Q_0} = 4\, C_2 \frac{x}{D_0} \qquad (30)$$

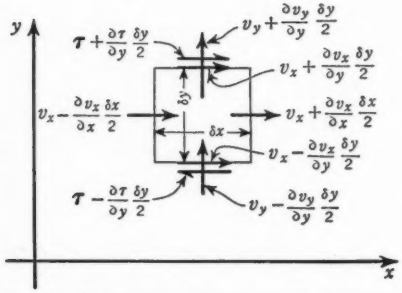
$$\underset{r\to\infty}{\text{Limit}}\, \frac{v_r}{v_0}\frac{r}{D_0} = -\frac{C_2}{2} \qquad (31)$$

and

$$\frac{E}{E_0} = \frac{1}{3\,C_2}\frac{D_0}{x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

Characteristics of the Mixing Process.—The foregoing analysis of the pattern of mean flow was made with complete disregard for the detailed characteristics of the diffusion pattern on which the phenomenon depends. If the assumed distribution of mean velocity and the resulting diffusion rate are in even approximate agreement with actual flow conditions, however, it should be possible to obtain at least a rough indication of the diffusion characteristics.

Since the pressure distribution was assumed to be hydrostatic, the only force to be considered in analyzing the interaction of the jet and the surrounding fluid must be the shear exerted along any longitudinal plane. The cumulative effect of such shear will necessarily disappear as the distance from the jet axis becomes great (that is, a zero net force corresponds to a zero longitudinal gradient of the total momentum flux), but within the zone of diffusion the intensity of shear τ may be expected to vary considerably. In fact, its local magnitude may be evaluated in terms of the change in momentum flux which occurs as fluid passes through an elementary part of the diffusion zone.


y
x
$\tau + \frac{\partial \tau}{\partial y}\frac{\delta y}{2}$
$v_y + \frac{\partial v_y}{\partial y}\frac{\delta y}{2}$
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δy
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$v_x - \frac{\partial v_x}{\partial y}\frac{\delta y}{2}$
$\tau - \frac{\partial \tau}{\partial y}\frac{\delta y}{2}$
$v_y - \frac{\partial v_y}{\partial y}\frac{\delta y}{2}$

FIG. 5.—DEFINITION SKETCH FOR ELEMENTARY SHEAR AND MOMENTUM FLUX

With reference to Fig. 5, which represents the longitudinal cross section of a cubic element of space within the expanding jet, the flux of longitudinal momentum may be expressed as the product of the elementary volume flux at right angles to any face and of the corresponding longitudinal component of momentum per unit volume. The net longitudinal component of force on the faces of the element should then equal the net flux of longitudinal momentum through all faces of the element. For two-dimensional flow (that is, zero longitudinal shear on the $x-y$ faces), the momentum equation will be

$$\left(\tau+\frac{\partial \tau}{\partial y}\frac{\delta y}{2}\right)\delta x-\left(\tau-\frac{\partial \tau}{\partial y}\frac{\delta y}{2}\right)\delta x=\rho\left(v_x+\frac{\partial v_x}{\partial x}\frac{\delta x}{2}\right)^2\delta y-\rho\left(v_x-\frac{\partial v_x}{\partial x}\frac{\delta x}{2}\right)^2\delta y$$
$$+\rho\left(v_x+\frac{\partial v_x}{\partial y}\frac{\delta y}{2}\right)\left(v_y+\frac{\partial v_y}{\partial y}\frac{\delta y}{2}\right)\delta x-\rho\left(v_x-\frac{\partial v_x}{\partial y}\frac{\delta y}{2}\right)\left(v_y-\frac{\partial v_y}{\partial y}\frac{\delta y}{2}\right)\delta x \ldots (33)$$

which reduces to the following statement of conditions at a point:

$$\frac{\partial \tau}{\partial y} = \rho\left(v_x\frac{\partial v_x}{\partial x}+v_y\frac{\partial v_x}{\partial y}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

Evidently, the local intensity of shear will involve the integral of Eq. 34 with

respect to y from the axis to the point in question:

$$\tau = \rho \int_0^y \left(v_x \frac{\partial v_x}{\partial x} + v_y \frac{\partial v_x}{\partial y} \right) dy \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

Because of the difficulty of expressing v_y by other than graphical methods, evaluation of this relationship must also proceed graphically, the results being shown in plotted form subsequently. It is at once possible, on the other hand, to express certain characteristics of the turbulence in terms of this function.

From the Reynolds equations of acceleration, the intensity of longitudinal shear at any point in a turbulent fluid may be written as the sum of a mean viscous stress and an apparent turbulent stress:

$$\tau = \mu \frac{\partial v_x}{\partial y} - \rho \overline{v'_x v'_y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

in which μ is the dynamic viscosity of the fluid. Under conditions of such pronounced mixing as exists within the eddy region of a spreading jet, it appears reasonable to presume that the term for mean viscous shear will be relatively unimportant. Indeed, this was prerequisite to the basic assumption of dynamic similarity. The necessarily finite magnitude of the mean product of the turbulent velocity components $\overline{v'_x v'_y}$, on the other hand, indicates that a finite degree of correlation must exist between the components v'_x and v'_y. The turbulence must be definitely nonisotropic, therefore, and in all probability the root-mean-square values of these components will differ from one another at every point. Because of the arbitrary nature of the assumed mean velocity distribution, it cannot be expected that the resulting mean flow pattern will yield any useful clue as to the relative magnitudes of these root-mean-square terms, and as a rough approximation it must necessarily be assumed that they are proportional. Further assuming that the Prandtl concept of proportionality between these components and the product of a so-called mixing length l and the mean velocity gradient is not greatly in error, it follows that, qualitatively,

$$\tau = \rho \sqrt{\overline{(v'_y)^2}}\, l \frac{\partial v_x}{\partial y} = \rho\, l^2 \left(\frac{\partial v_x}{\partial y} \right) \left| \frac{\partial v_x}{\partial y} \right| \quad \ldots\ldots\ldots\ldots\ldots (37)$$

in which the product $\sqrt{\overline{(v'_y)^2}}\, l$ represents the diffusion coefficient ϵ (that is, the so-called kinematic eddy viscosity). Once Eq. 35 has been evaluated, it should be possible to express the approximate variation of l, $\sqrt{\overline{(v'_y)^2}}$, and ϵ as dimensionless functions of coordinate space, just as in the case of such other variables as v, Q, and E. The results are shown graphically in the following section.

Experimental Results

Equipment and Test Procedure.—An experimental program resulting from a wartime fog-dispersal project of the Iowa Institute of Hydraulic Research (at the State University of Iowa, Iowa City, Iowa) involved the measurement of the mean velocity distribution in both two-dimensional and three-dimen-

sional jets over a considerable range of each independent variable. Purely for convenience, all measurements were made in air, but there is no reason to presume that comparable results would not be obtained in any other fluid—whether liquid or gas—of moderately low viscosity. Essentially the same

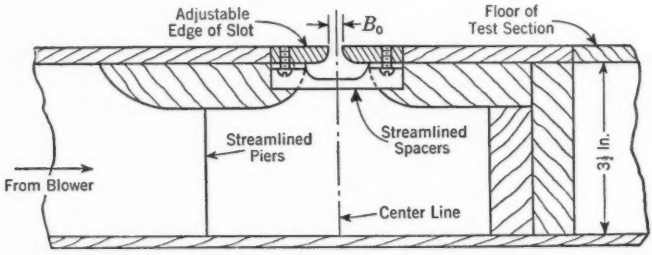

FIG. 6.—DETAILS OF SLOT AND APPROACH

experimental equipment was used in each phase of the project, the flow being produced by a 2-hp centrifugal fan with suitable ducts and stilling devices. The two-dimensional slot (Fig. 6) was formed from two 6-ft strips of $1\frac{1}{4}$-in. by $\frac{3}{8}$-in. cold-rolled steel, carefully rounded along the inner edges and accurately adjustable in spacing from $\frac{1}{32}$ in. to $\frac{1}{4}$ in. This slot was set flush with the surface of a 6-ft by 12-ft level floor, and was bounded at the ends by 6-ft by 12-ft vertical walls. (Because attempts to measure the velocity distribution in the zone of establishment for even the $\frac{1}{4}$-in. slot proved futile, a special 1-in. slot

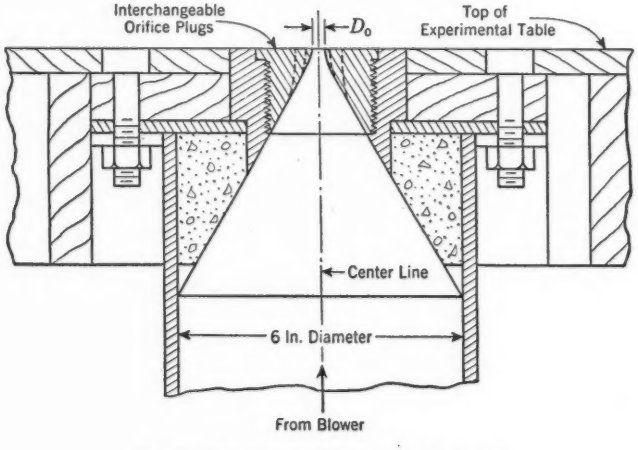

FIG. 7.—DETAILS OF ORIFICE AND APPROACH

was made for this purpose.) The circular orifice (Fig. 7) was set into the top of a 4-ft by 4-ft table, and consisted of three interchangeable brass sections having outlet diameters of $\frac{1}{4}$ in., $\frac{1}{2}$ in., and 1 in. with well-rounded transitions from the same 60° convergent approach section. In both cases, gage carriages

permitted motion of measuring instruments in three coordinate directions, scales with verniers which read to 0.001 ft indicating the coordinate position.

Because of the extreme variation in velocity with location, it was necessary to utilize several different methods of velocity measurement. The efflux velocity of the jet and the velocity distribution in the outlet vicinity were determined with a No. 20 hypodermic needle used as a stagnation tube and connected directly to a differential water manometer read by point gage to 0.001 ft. In zones of moderate velocity the water manometer was replaced by a

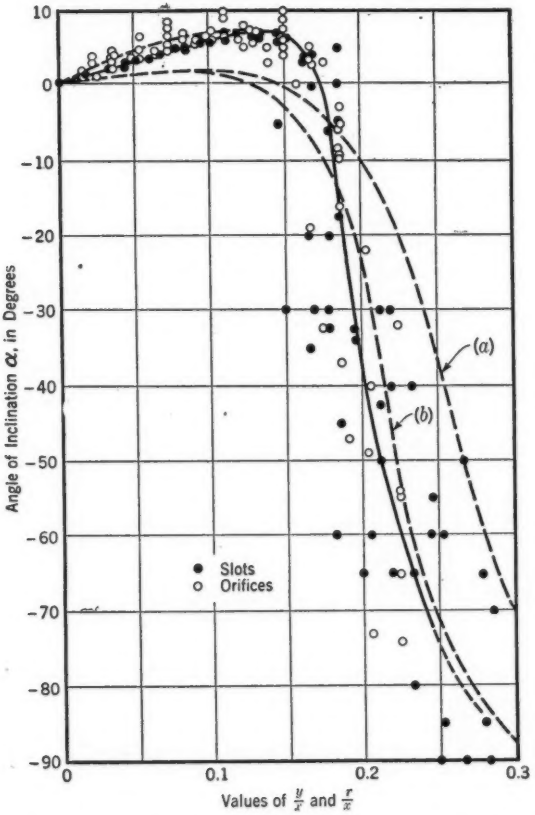

FIG. 8.—VARIATION IN DIRECTION OF FLOW WITHIN JETS FROM BOTH SLOTS AND ORIFICES

sensitive two-liquid gage, which was read by micrometer to 0.001 in. of alcohol. The lowest velocities were measured by a midget spoke-vane anemometer with jeweled bearings, which had been calibrated in an air tunnel. Except in the zones of flow establishment, the velocity instruments were oriented with respect to flow direction in accordance with prior indications of a directional vane consisting of a thin plate with a stagnation tube mounted on each side a short distance behind the leading edge. The vane was rotated about an axis through the leading edge until a zero differential reading was obtained on a manometer

connected to the two stagnation tubes. In zones of very low velocity the vane was replaced by a fine jet of smoke for visual observation.

Since the distribution of velocity at any cross section was assumed to be governed solely by the efflux velocity, the outlet dimension, and the longitudinal distance to the section, these three independent variables were generally so controlled in the tests that the maximum practicable range of each was investigated in systematic steps as the other two were held constant. There were three exceptions to this general procedure: First, because the zone of established flow was of primary interest, only a few exploratory measurements of necessarily limited range were made in the zone of establishment. Second, as a result of the evident similarity of the distribution curves at all sections of the established flow, only a sufficient number of direction measurements (see Fig. 8) were made to establish the approximate variation of vector inclination for the two-dimensional and the three-dimensional cases beyond reasonable

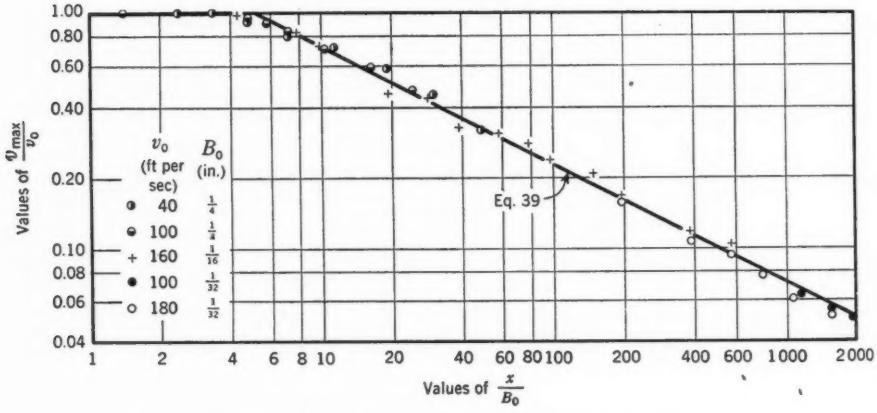

FIG. 9.—DISTRIBUTION OF CENTER-LINE VELOCITY FOR FLOW FROM SLOT

doubt, and this was then used to orient the velocity instruments for all subsequent measurements. Third, in order to determine the variation in v_{max} over the greatest possible range, independent center-line measurements were made far beyond the region in which the complete lateral distribution could accurately be determined.

Results for the Two-Dimensional Jet.—Measurements of the center-line velocity from the efflux section over a longitudinal distance equal to several thousand times the slot width are shown as a dimensionless logarithmic plot in Fig. 9. Except for a limited zone of transition, the experimental points for all slot widths and efflux velocities follow closely either a horizontal line with the ordinate magnitude of unity for the zone of flow establishment, or a 1:2 sloping line for the zone of established flow. Since these two lines intersect at the point $x_0/B_0 = 5.2$, it follows at once that the constant C_1 for the equations of the zone of establishment of any two-dimensional jet will have the magnitude $C_1 = B_0/(\sqrt{\pi}\, x_0) = 0.109$.

Introduction of this value into Eq. 12 yields the specific relationship for the velocity distribution in the diffusion region of the two-dimensional zone of establishment

$$\log_{10}\frac{v_x}{v_0} = -18.4\left(0.096 + \frac{y - B_0/2}{x}\right)^2 \quad \ldots\ldots\ldots\ldots (38)$$

which is plotted in Fig. 10, using the lateral location of the slot boundary as the zero reference on the ordinate scale. Superposed on the analytical curve are the measured values obtained in the exploratory study of the velocity distribution in this zone. The agreement between analysis and experiment is quite satisfactory.

For the zone of established flow, introduction of the same experimental coefficient $C_1 = 0.109$ into Eq. 23 leads to the specific expression for the sloping line of Fig. 9,

$$\frac{v_{max}}{v_0}\sqrt{\frac{x}{B_0}} = 2.28 \ldots (39)$$

Eq. 39 represents the limiting form of Eq. 24 after introduction of the constant C_1:

$$\log_{10}\frac{v_x}{v_0}\sqrt{\frac{x}{B_0}} = 0.36 - 1.84\frac{y^2}{x^2} \ldots (40)$$

Eq. 40 is plotted in Fig. 11, together with all measured values for the longitudinal velocity component. Despite appreciable experimental scatter, the analytical curve is in reasonable agreement with the measurements over the entire range of the independent variables. The graphical solution of the continuity relationship for v_y, in terms of the constant C_1, is shown in the same general manner in Fig. 12, together with experimental values obtained by multiplying the measured velocities by the sines of the corresponding angles of inclination of Fig. 8. Although the data follow the general trend of the theoretical curve, the values computed from the measurements are at least 100% greater than those indicated by the plotted function—the discrepancy also

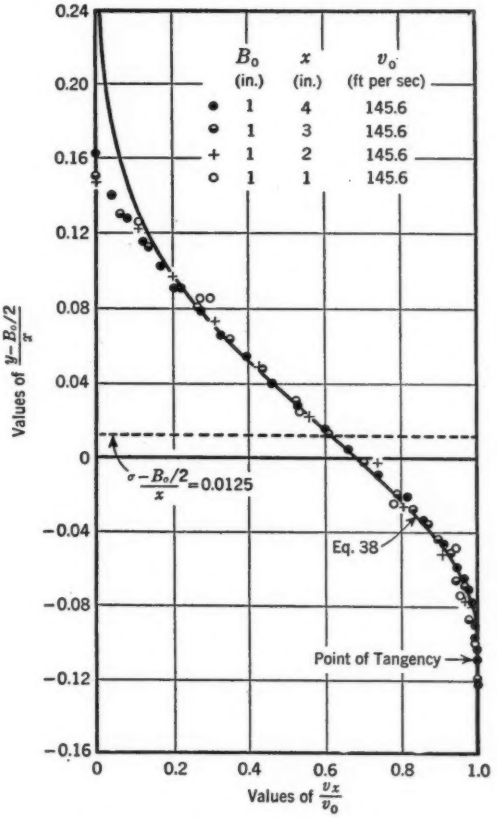

FIG. 10.—DISTRIBUTION OF LONGITUDINAL VELOCITY IN ZONE OF ESTABLISHMENT OF FLOW FROM SLOTS

being apparent from comparison of the analytical curve (curve (*a*)) of Fig. 8 with the measured values. No explanation can be given for this disagreement; neither do the computations disclose any likelihood of mathematical error, nor have repeated measurements by different observers led to essentially different

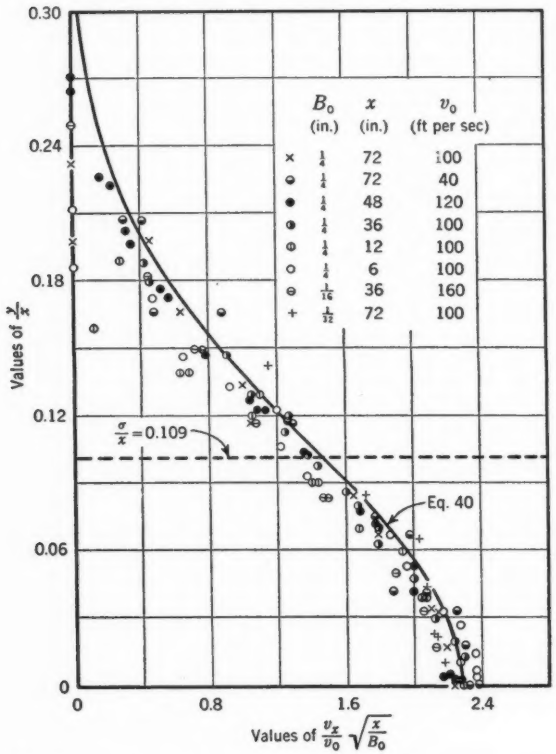

FIG. 11.—DISTRIBUTION OF LONGITUDINAL VELOCITY IN ZONE OF ESTABLISHED FLOW FROM SLOTS

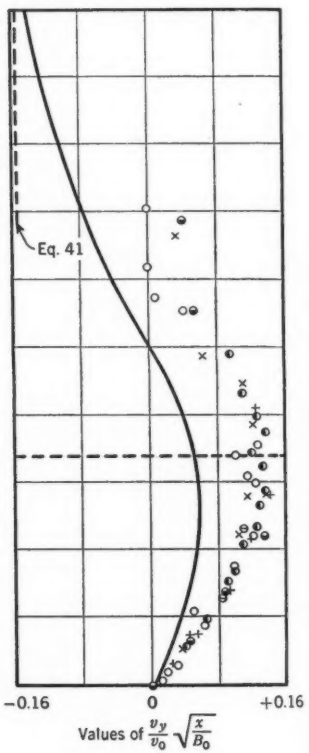

FIG. 12.—DISTRIBUTION OF LATERAL VELOCITY IN ZONE OF ESTABLISHED FLOW FROM SLOTS

results. Fortunately, the angles involved were so small as to result in a negligible error in the longitudinal velocity. Because of the lack of dependable velocity measurements in the outer region, it cannot be stated whether or not the experimental data approach the velocity-of-entrainment limit given by the following specific form of Eq. 26:

$$\operatorname*{Limit}_{y\to\infty} \frac{v_y}{v_0}\sqrt{\frac{x}{B_0}} = -0.155 \quad \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

Eqs. 13 and 15 for the volume-flux ratio and the energy-flux ratio in the zone of flow establishment become, in terms of the constant C_1,

$$\frac{Q}{Q_0} = 1 + 0.080\,\frac{x}{B_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

and

$$\frac{E}{E_0} = 1 - 0.035\,\frac{x}{B_0} \qquad (43)$$

Likewise, Eqs. 25 and 27 for the zone of established flow will reduce to

$$\frac{Q}{Q_0} = 0.62\,\sqrt{\frac{x}{B_0}} \qquad (44)$$

and

$$\frac{E}{E_0} = -\,1.86\,\sqrt{\frac{B_0}{x}} \qquad (45)$$

Eqs. 42, 43, 44, and 45 (as well as the basic expression $M/M_0 = 1$) are plotted in Fig. 13, together with values obtained by integration of the measured velocity-distribution curves for typical runs with x as the independent variable.

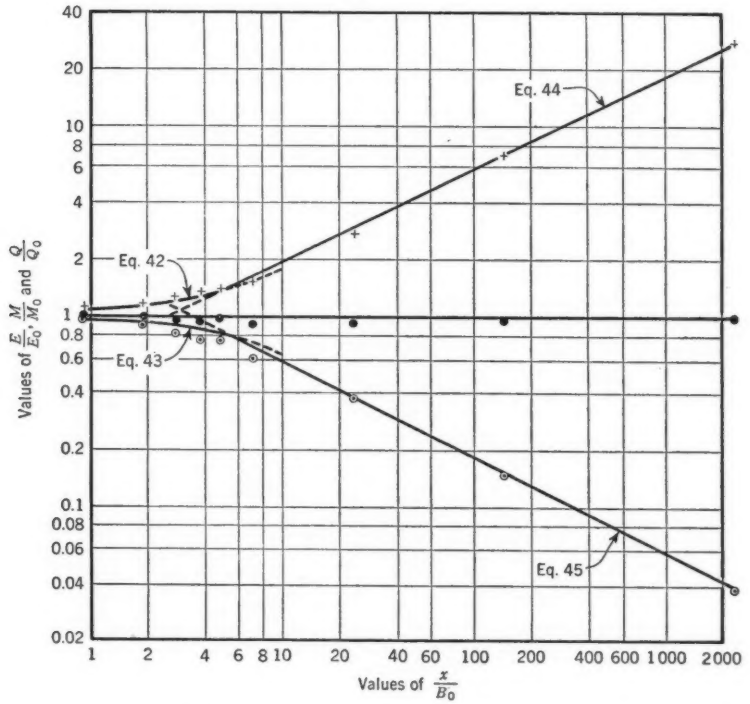

FIG. 13.—DISTRIBUTION OF VOLUME, MOMENTUM, AND ENERGY FLUX DOWNSTREAM FROM SLOT

Although no measurements of turbulence were made in this experimental program, it is nonetheless pertinent to note the results of the approximate analysis of the eddy pattern. Fig. 14 includes the simultaneous graphical solutions of Eqs. 35 and 37 as dimensionless curves for τ, $\sqrt{\overline{(v'_y)^2}}$, l, and ϵ at any

section of the zone of established flow, the assumed curve for v_x being included for purposes of orientation. As will be noted therein, the zone of maximum intensity of shear, and hence maximum intensity of turbulence, lies somewhat inside the point of inflection or maximum slope of the velocity curve (that is, where $y = \sigma$). The scale of the eddies is a minimum in this general zone, whereas the mixing coefficient attains a maximum at the center line. The indications are, of course, merely qualitative, but nevertheless quite significant.

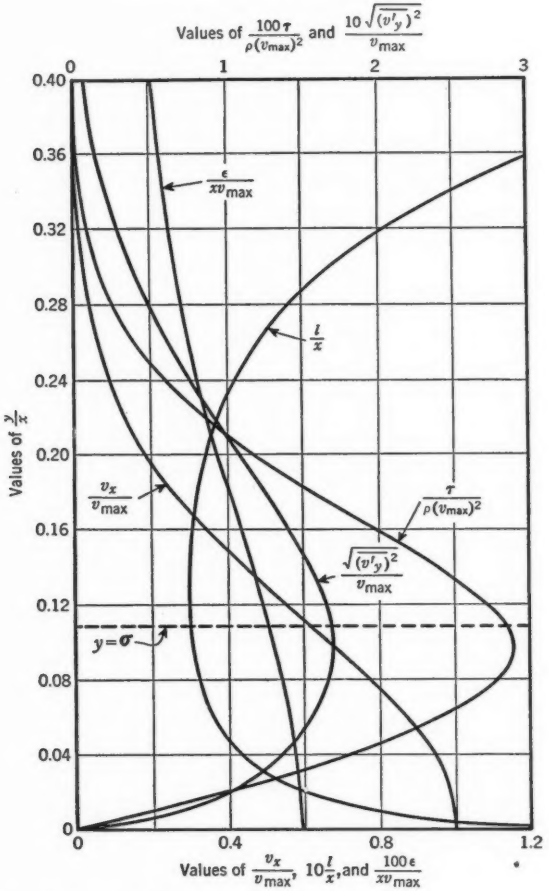

FIG. 14.—APPROXIMATE DISTRIBUTION OF TURBULENCE CHARACTERISTICS IN TWO-DIMENSIONAL ZONE OF ESTABLISHED FLOW

Results for the Three-Dimensional Jet.—As in the two-dimensional case just discussed, a logarithmic plot of all center-line velocity measurements (Fig. 15) yields a horizontal and a sloping line for the two zones of flow, that for the established zone now having a 1:1 slope. The intersection of these lines at the point $x_0/D_0 = 6.2$ at once yields $C_2 = D_0/(2\,x_0) = 0.081$. In

terms of this constant, Eq. 19 for the distribution of the longitudinal velocity component in the zone of establishment assumes the specific form:

$$\log_{10}\frac{v_x}{v_0} = -33\left(0.081 + \frac{r - D_0/2}{x}\right)^2 \quad \ldots\ldots\ldots\ldots (46)$$

which is plotted, together with measured data, in Fig. 16. Unlike the case of the slot, the agreement between analysis and measurement is not wholly satisfactory. As shown by the broken line, considerable improvement would result through use of the smaller border value $x_0/D_0 = 4.5$ (that is, $C = 0.111$).

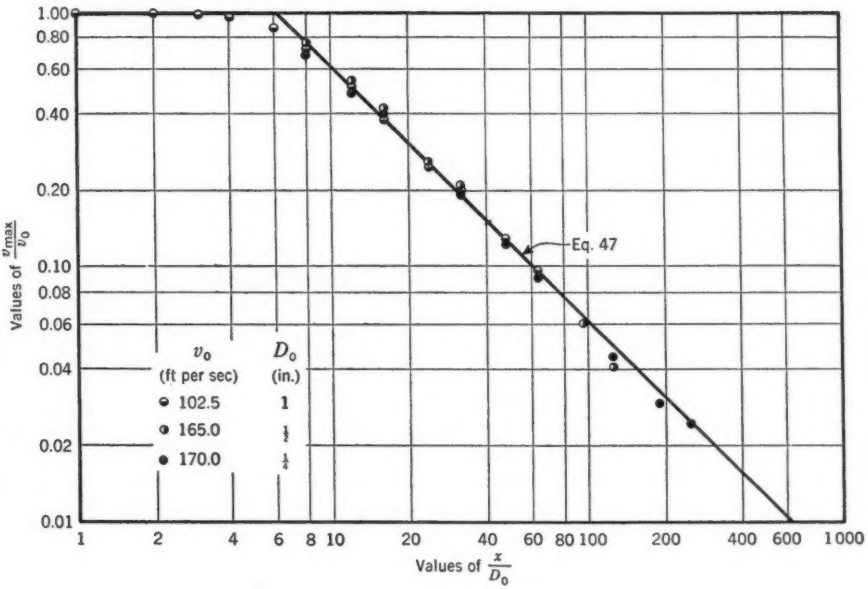

FIG. 15.—DISTRIBUTION OF CENTER-LINE VELOCITY FOR FLOW FROM ORIFICE

However, since close agreement in the limited zone of establishment is considered less significant than evaluation of both zones (as in the two-dimensional case) in terms of the same coefficient, the value $C_2 = 0.081$ is used throughout. Thus, Eq. 28 is seen to take the specific form:

$$\frac{v_{max}}{v_0}\frac{x}{D_0} = 6.2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

which corresponds to the sloping line of Fig. 15. This is simply the limiting form of the general relationship for v_x of Eq. 29, which accordingly becomes

$$\log_{10}\frac{v_x}{v_0}\frac{x}{D_0} = 0.79 - 33\frac{r^2}{x^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (48)$$

and is plotted in Fig. 17, together with measured data. Despite considerable experimental scatter due to the great range of the independent variables,

the points will be seen to follow the probability curve with good approximation. Fig. 18 gives the graphical solution of the continuity equation for v_r, supplemented by experimental results obtained through multiplying the measured vector magnitudes by the sines of the corresponding angles of inclination. As in the two-dimensional case, the data follow the trend of the

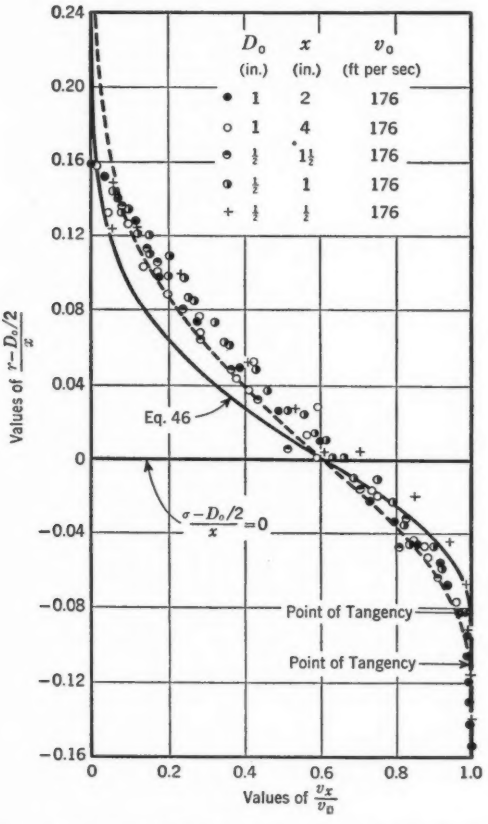

FIG. 16.—DISTRIBUTION OF LONGITUDINAL VELOCITY IN ZONE OF ESTABLISHMENT OF FLOW FROM ORIFICES

theoretical curve, but the values computed from the velocity measurements are at least twice as great as those of the plotted function (see also curve (*b*), Fig. 8). It is evidently impossible to discover whether or not they finally approach the hyperbolic curve corresponding to the following specific form of Eq. 31 for the radial velocity of entrainment:

$$\operatorname*{Limit}_{r\to\infty} \frac{v_r}{v_0}\frac{x}{D_0} = -0.040\frac{x}{r} \qquad (49)$$

Introduction of the experimentally determined coefficients in Eqs. 20 and 22 for the volume-flux and energy-flux ratios in the zone of flow establishment

gives

$$\frac{Q}{Q_0} = 1 + 0.083\,\frac{x}{D_0} + 0.0128\,\frac{x^2}{(D_0)^2} \quad \ldots\ldots(50)$$

and

$$\frac{E}{E_0} = 1 - 0.090\,\frac{x}{D_0} - 0.029\,\frac{x^2}{(D_0)^2} \quad \ldots\ldots(51)$$

Similar evaluation of Eqs. 30 and 32 for the zone of established flow yields

$$\frac{Q}{Q_0} = 0.32\,\frac{x}{D_0} \quad \ldots\ldots(52)$$

and

$$\frac{E}{E_0} = 4.1\,\frac{D_0}{x} \quad \ldots\ldots(53)$$

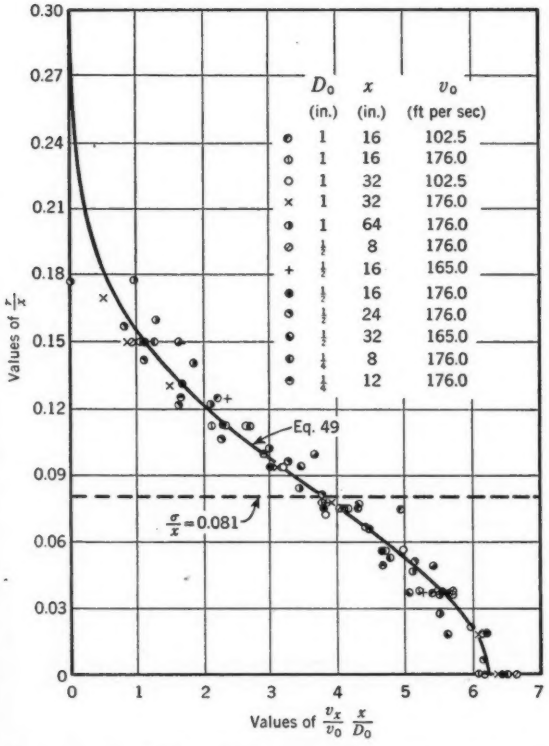

FIG. 17.—DISTRIBUTION OF LONGITUDINAL VELOCITY IN ZONE OF ESTABLISHED FLOW FROM ORIFICES

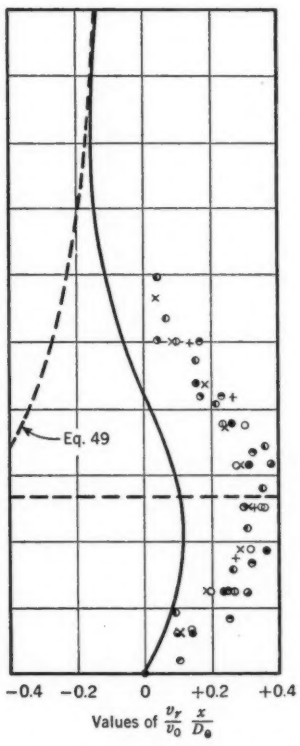

FIG. 18.—DISTRIBUTION OF RADIAL VELOCITY IN ZONE OF ESTABLISHED FLOW FROM ORIFICES

Eqs. 50, 51, 52, and 53 are plotted in Fig. 19, together with points representing the corresponding integrals of measured velocity-distribution curves for typical runs in which x was the only independent variable.

Except for the fact that the development leading to Eq. 35 must be revised according to the distinction between the cubic element of the two-dimensional case and the cylindrical element of the three-dimensional case, evaluation of the approximate characteristics of the turbulence proceeds in essentially the same manner. Solution of the corresponding differential equations results in the functions for τ, $\sqrt{\overline{(v'_\tau)^2}}$, l, and ϵ plotted in Fig. 20 in dimensionless form together with the function for v_x for purposes of orientation. Inasmuch as the

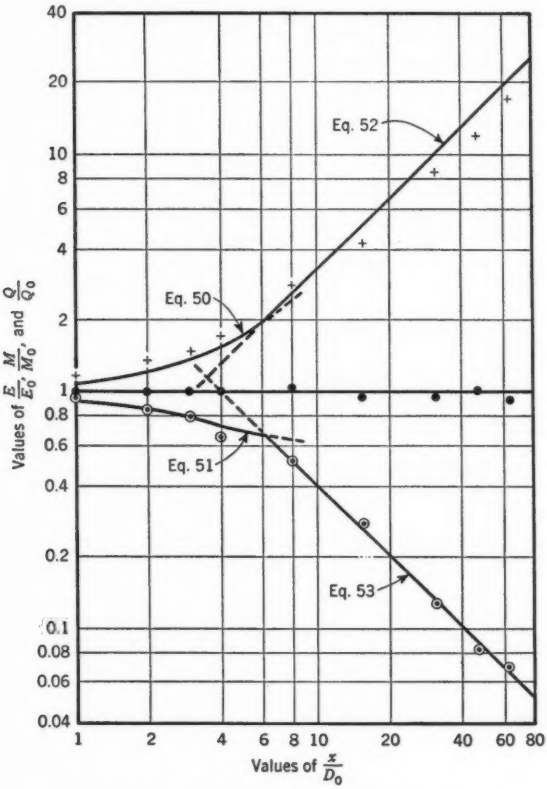

FIG. 19.—DISTRIBUTION OF VOLUME, MOMENTUM, AND ENERGY FLUX DOWNSTREAM FROM ORIFICE

curves differ from those of Fig. 14 only in minor details, the same comments are again applicable.

Generalization of the Mean Flow Pattern.—Although the results of the combined analytical and experimental investigation presented in the foregoing must be considered as approximate rather than as rigorous, the series of specific equations and plotted curves are nevertheless sufficiently close to actual fact to be used with confidence in general problems of design. The equations or diagrams themselves indicate no limit of applicability. In fact, no systematic deviation from the respective functions was noted with change in either length

or velocity scale. Two probable limits of applicability should therefore be mentioned. At great distances from the efflux section—however accurately the measured curves may be extrapolated—the velocities of flow become so low that extraneous effects rather than the dynamics of the jet itself will begin to govern the flow pattern. Although no significant deviation of the center-line velocities from the predicted function was found in experiments at distances

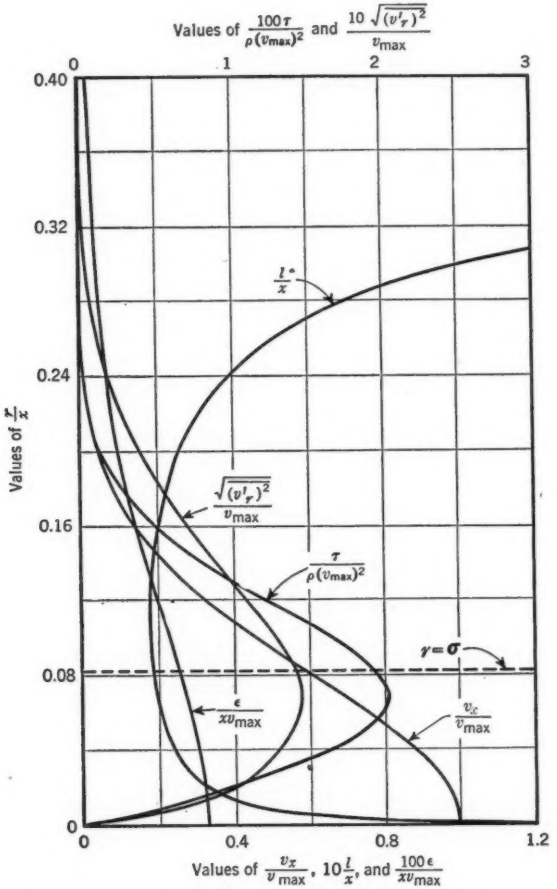

FIG. 20.—APPROXIMATE DISTRIBUTION OF TURBULENCE CHARACTERISTICS IN THREE-DIMENSIONAL ZONE OF ESTABLISHED FLOW

from the efflux section equal to several hundred times the orifice diameter and to several thousand times the slot width, it must be emphasized that extreme care was taken to eliminate all possible disturbance of the jet. At low Reynolds numbers for the efflux section, moreover, it is to be expected that viscous influences will prevent the full development of turbulence in the region of diffusion. The lowest Reynolds number, $\mathbf{R} = v_0 L_0/\nu$, attained in the tests described herein was about 1,500. There is, of course, no reason to expect any

difference between the flow of a gas and the flow of a liquid at comparable Reynolds numbers. The results may be considered applicable, therefore, to the efflux of either type of fluid—provided only that gas velocities do not become so high that acoustic effects begin to influence the flow pattern and that liquid velocities do not become so high that appreciable cavitation occurs within the eddies.

If a streamline or stream surface of the mean motion is defined as a line or surface (depending on whether the flow is two-dimensional or three-dimensional) across which no flow takes place, it should be possible from the foregoing results to develop—either from the velocity-distribution curves or from the stream function corresponding to these curves—a systematic sequence of such lines or surfaces showing at a glance the pattern of mean motion in both the zone of

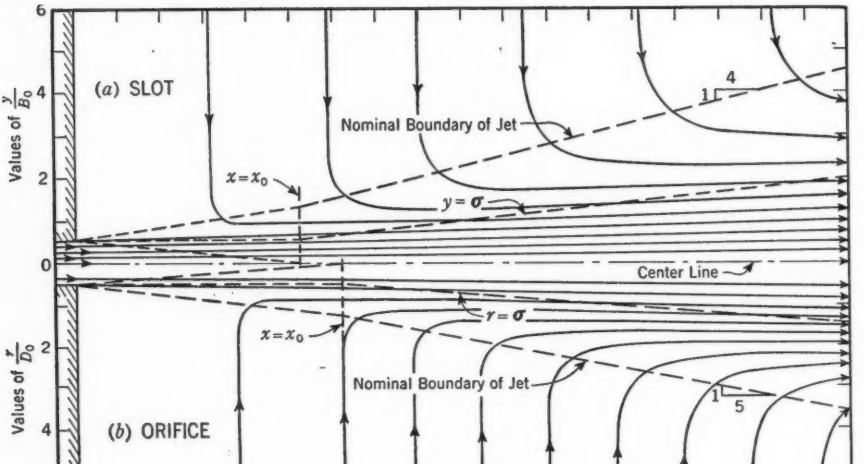

FIG. 21.—GENERAL PATTERNS OF FLOW FROM SLOT AND ORIFICE

establishment and the zone of established flow. Such a diagram is presented in Fig. 21, in which the upper half corresponds to efflux from a long slot and the lower half to that from a circular orifice. In the two-dimensional case, the spacing of the streamlines is inversely proportional to the local magnitude of the velocity, so that the speed as well as the direction of flow is readily apparent. In the three-dimensional case the spacing of the stream surfaces is inversely proportional to the velocity times the radial distance to the point in question, so that—although the direction of motion is immediately evident—comparison of velocity magnitudes by eye is not readily possible. Each of the diagrams contains the locus of the point of inflection (that is, the maximum slope) of the error curve for the longitudinal velocity distribution, which corre-

sponds roughly to the region of maximum intensities of shear and turbulence. Also indicated is the nominal boundary of the jet—that is, the zone in which the longitudinal velocity is only a small percentage of that at the center line. The nominal boundary of the jet from a slot will thus have a slope of approximately 1:4, whereas that for an orifice is approximately 1:5. As previously indicated, this diagram should be characteristic of all jets under comparable flow conditions.

Not until after the completion of the present paper did the authors receive a copy of the very excellent and comprehensive treatise by H. Reichardt,[12] which discusses submerged jets as one of a series of related diffusion phenomena. Noteworthy in connection with the pattern of mean flow just presented is Mr. Reichardt's conclusion that the lateral streamlines of the two-dimensional jet—in contradistinction to the radial streamlines of the three-dimensional jet (and to the lateral streamlines as shown at the top of Fig. 21)—must be asymptotic to a family of parabolas having a common focus at the origin of the coordinate system, in order that the velocity of the lateral flow, like that of the radial flow, may properly be zero at infinity. This would accordingly modify the indicated outskirts of the two-dimensional pattern, but would affect the streamlines of the central flow to a wholly imperceptible degree.

Conclusions

Through assumptions of hydrostatic pressure distribution, dynamic similarity of the diffusion process, and normal-probability variation of the velocity, analytical expressions have been developed for the patterns of mean flow within submerged jets from both slots and orifices. Measurements conducted on the flow of air at different velocities from slots and orifices of different sizes have been shown to be in substantial agreement with the analytical relationships, and have provided the single experimental coefficient required to complete the analysis in the two-dimensional and in the three-dimensional case. Algebraic expressions and plotted curves have been presented in dimensionless form for the distribution of velocity, volume flux, and energy flux in the zone of flow establishment and the zone of established flow for each boundary condition. In addition, a qualitative treatment of the variation in mixing characteristics and a generalized diagram of the mean flow pattern have been given. All results are considered applicable for design purposes to the flow of any liquid or gas at moderate to high Reynolds numbers.

Acknowledgments

This paper represents an original contribution of the Iowa Institute of Hydraulic Research of the State University of Iowa, all measurements having been conducted in the Institute laboratories and all analytical developments having been carried to completion by members of the Institute staff. The experiments on efflux from slots—with the exception of measurements in the zone of flow establishment contributed by D. E. Metzler—were performed by

[12] "Gesetzmässigkeiten der freien Turbulenz," by H. Reichardt, *Forschungsheft 414*, Verein Deutscher Ingenieure, 1942.

Messrs. Albertson and Jensen as a seminar project in the Department of Mechanics and Hydraulics of the University. The experiments on efflux from circular orifices were performed by Mr. Dai as a thesis investigation in partial fulfilment of the requirements for the degree of Master of Science in the Graduate College of the University. Because of its direct application to certain studies being conducted by the Institute for the David Taylor Model Basin (Carderock, Md.), Bureau of Ships, United States Navy, part of the analysis presented herein was supported by Navy research funds. J. S. McNown, Assoc. M. ASCE, acted frequently as consultant during the preparation of the paper, and W. D. Baines, Jun. ASCE, evaluated many of the more complex integrals. All phases of the project were under the direct supervision of Professor Rouse.

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PAPERS

THE RELATION OF URBAN EXPRESSWAYS TO MASS TRANSPORTATION FACILITIES

BY CURTIS G. BRADFIELD,[1] M. ASCE

SYNOPSIS

Although the provision of urban expressway networks to accommodate all classes of metropolitan vehicular traffic would be a much desired luxury, such construction for other than intercity traffic appears economically unjustifiable. The fact that more than half of the local traffic is unnecessary and could be eliminated by modernizing mass transportation facilities is emphasized. The irreducible part of local traffic could be materially aided in many old cities by correcting inconsistencies in street layouts and establishing reasonable continuity of traffic flow. The pattern for justifiably modernized off-the-surface rapid transit nuclear systems is described and the desirability of extending commuter service into the central business district is recommended.

INTRODUCTION

Almost every major city in the United States is being urged to undertake the construction of elaborate urban systems of limited access expressways designed largely for the accommodation of local motorists. Such systems will provide: (*a*) Inner, intermediate, and outer circumferential belt lines; (*b*) radial arterial connections between these loops; and (*c*) continuous flow interchanges at the junction points.

Vast sums of the taxpayers' money will be required to create these facilities. For example, the city and the adjacent county of Los Angeles, Calif., are contemplating an over-all expressway plan estimated to cost $582,000,000. More compact cities can obtain equivalent coverage for somewhat smaller, but still staggering, sums. In addition, these limited access freeways, particularly when depressed, entail the sacrifice of large strips of originally tax-producing private property and thus will add still more to the remaining taxpayers' burdens.

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1949.**

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Nevertheless, although gregarious man likes crowds and busy streets, it is obvious to everyone that traffic conditions in metropolitan areas have become intolerable and some remedy must be found. Apparently there are two basic alternatives. One is to provide the urban expressways and thus to perpetuate the usage of the personal automobile as an uneconomical, illogical substitute for rapid transit facilities; the other, to discourage the unnecessary use of the automobile and to provide, instead, modernized, adequate mass transportation.

The Traffic Problems

In most cities the business district traffic, in reality, presents three problems—first, the intercity traffic following a natural geographic route through the very heart of the "downtown" section which these old highways brought into being; second, the local traffic overcrowding the few available through streets; and, third, mass transportation equipment, such as streetcars, buses, etc., operating almost end to end on the surface of the streets. Each of these phases of the over-all problem has sharply distinctive characteristics. A thorough diagnosis of the malady does not result from merely lumping these problems together and prescribing the urban expressway as the panacea for all these ills.

Instead, each of the traffic types should be analyzed separately. The degree and the extent of actual necessity of each category must be determined and, once they are established, the kind and the extent of relief the city can justify and afford to provide becomes apparent.

Relief for Intercity Traffic

The intercity and, as a subproblem with somewhat similar characteristics, the cross-the-city traffic are types that, obviously, cannot be reduced. In fact, most cities want to encourage such traffic because the ever-increasing volume of heavy commercial vehicles—trucks, trailers, semitrailers, and the like—is as much the lifeblood of trade as any railroad or waterway ever was. Also, that part of intercity traffic consisting of private motorcars should not be shunned, for it is a well-known phenomenon of merchandising that an appreciable amount of trade does result from a stream of travelers passing through a city. Consequently, it would be shortsighted indeed for a community deliberately to discourage the tourist from entering a town.

Nevertheless, although the trucker must make contact with the typical truck-loading exchange terminals usually located on the fringe of the central business district, and the tourist should penetrate a city deep enough to be within reach of hotel, restaurant, and shop, their presence on the old highway lines invariably meandering through the exact center of the business district is certainly a traffic congesting element that is unnecessary and unjustifiable in that particular zone. Therefore, where this type of traffic exists in a volume sufficient to constitute a major traffic problem, the indicated remedy is to remove it from the surface of the city streets.

For intercity and cross-the-city traffic problems, there is little doubt but that the urban expressway is the logical facility. The limited access highway linking city with city has amply demonstrated its worth, and the extension

of such construction on through a city, as either a depressed or an elevated expressway, can usually be justified. Such a facility frequently can be routed so as to bend away from the original highway line just sufficiently to pass along the edge of the active central district. When so installed, it will still give access to the business zone and adequately serve the intercity and satellite community traffic terminating and originating in the metropolis, the through traffic (which, although proportionately small numerically, consists largely of highly objectionable heavy trucks, semitrailers, and the like), and the traffic which enters the city from an outlying community for a destination in the city beyond the central business district. In most cities, only a few blocks separate highly valuable property on the edge of the central business district from the deteriorated "fringe" section of low value. If the intercity expressways are located in this fringe, large construction economies will be achieved.

Relief for Local Traffic

Although the urban expressway is the indicated remedy for the foregoing class of traffic, the projection of this theory to justify similar installations for substantially local traffic flowing to and fro between the edges of a city and the perimeter of the central business district is seemingly unwarranted. In fact, in the second, or local traffic phase of the over-all problem, an entirely different basic condition prevails. In this case, most of the traffic is not unavoidable and it is not at all certain that it should be encouraged.

An origin-and-destination traffic survey conducted in a large eastern city disclosed that, of every hundred people who brought their cars to a parking location in the central business district, seventeen were shopping, thirty were on business trips, and forty three were going to places of employment. For want of a better index, it can be assumed that these proportions are typical of all cities and it is obvious that a large part of such local traffic is probably not genuinely necessary. Nevertheless, this group of cars comprises the local traffic problem for which most of the in-city system of expressways is proposed —people going to work or to a business appointment.

Questions immediately arise. How much of this traffic is really mandatory? Is there not some less expensive method of transporting these people? Is it possible to continue sacrificing more and more of the valuable "downtown" areas for storage spaces for more and more automobiles as cities grow larger and larger? To demonstrate by exaggeration, if it is continually easier and easier for people to bring their cars into the business districts, eventually the central zones will be so filled with automobiles there will be no room left for people—or for business.

However, if complete urban expressway networks are provided, the well-known and most often stated criticisms are:

1. The inner circumferential expressway, in encircling the business district, will effectively fix the ultimate growth limits of that zone and may produce a throttling influence on future expansion. It is a well-known phenomenon of city development that a marked irregularity in the surface plane creates a psychological barrier and a restriction on normal growth. Cases in point are

Park Avenue (New York, N. Y.) before it was covered over, the Canal (Cincinnati, Ohio) before it was filled in, and the elevated trackage along Market Street (Philadelphia, Pa.), all of which were definite walls against urban development.

2. The arterial expressway radial connections between inner and intermediate circumferential belts will be built in the typical, obsolescent intermediate zones between downtown and suburban areas. These areas, usually from 1 mile to 3 miles wide, are in a transitory status and, actually, do not now present too much traffic congestion. However, by building expressways through these sections, it is likely that, in an average city, from 7 min to 12 min may be saved in driving from a suburb to the downtown district. Nevertheless, people have certain specific times for work or for business appointments and, consequently, will invariably use this saving in driving time for time out of the city—that is, they will leave home later when going to work. Therefore, almost everyone will arrive in, or leave from, the downtown area at approximately the same time.

3. As the inner circumferential expressway belt will only encircle, not penetrate, the downtown area, all local cars heading for a destination in the central business section will still be on the surface of the city streets while in the zone of most congestion.

4. To be consistent, if facilities are installed to expedite and encourage local traffic to come into the downtown area, tremendously improved parking or "terminal facilities," as the freeway proponents term them, must be provided to accommodate the cars while they are in the critical space area. Inasmuch as private interests have found little inducement to furnish such "terminals," it is likely that the taxpayer ultimately will also have to underwrite this construction.

Summing up, the net result of a huge investment in in-city expressways will be solely to get cars downtown a few minutes faster, only to add to substantially the same general confusion. Also, once downtown, what is to be done with the cars?

Assuredly, people are wanted downtown, but perhaps their cars are not wanted downtown after all. Therefore, the logical course of investigation is to first determine why people want to bring their cars into the central business district.

In almost every case, people elect to use a relatively expensive type of personal transportation because of the inadequacy of existing mass transportation. This inadequacy results from insufficient coverage, infrequent schedules, discomfort, noise, dirt, and all the other objectionable features so familiar to everyone and, also, from the fact that, since about 1920, the development of mass transportation has not kept pace with the movement volume of the people, because of the increasing use of the automobile for personal transportation.

When there was space in the central business districts to accommodate cars, it mattered little that one person after another discontinued using mass transportation and drove his automobile downtown. This procedure was not discouraged. On the contrary, it was encouraged. Streets were widened,

traffic control systems installed, parking garages built, and the taxpayers spent large sums of money to foster the use of the personal automobile. In so doing, the normal growth of mass transportation was stultified and its standards crystallized at the 1920 level.

It is readily apparent to any engineer that, from the standpoint of economics, the only way to move a volume of people from point to point is by mass transportation; that in big cities bulky, cumbersome, personal transportation probably can no longer be used in lieu of mass transportation; and that, whether cities expand horizontally or vertically, with increasing land values, downtown vehicular accommodations cannot continue to be expanded for the convenience of the 43% who merely drive to work and a sizable fraction of the 30% on business trips or the 17% who want to shop.

Therefore, before constructing extensive, expensive, systems of freeways to encourage the use of the private automobile further, it appears that the first obligation is to provide transportation in volume—to reverse the trend of the past and to encourage the development of mass transportation as a primary remedy for the local traffic phase of the over-all traffic problem. Just this result automatically will be accomplished if nothing is done about the local traffic situation. In fact, for all cities it could be stated that the time to do nothing about local traffic occurs when everyone clamors for something to be done—when it is clearly established that the breaking point in tolerance of traffic delays has been reached—for then the situation will begin to rectify itself. One by one, group by group, people will gradually discontinue driving their cars into the downtown area and will resort to the use of mass transportation facilities rather than personally endure the exasperation of operating a car in heavily congested traffic.

Of course, if left to chance, the mistake of 25 years is not going to be undone overnight. Although as more and more people resume using mass transportation, it, in turn, will gradually expand and modernize; such development, unless artificially accelerated, will require many years of slow progress and much more public clamoring to the various public service commissions. However, mass transportation is already recognized as one of the contributing phases of the traffic problem. Therefore, it should be possible to expedite the redevelopment of mass transportation materially at the same time such transportation is being corrected as a traffic problem.

Although improved mass transportation should substantially reduce the surface street congestion, there will always be that irreducible minimum of local traffic arising from cars that have to be brought into the cities. Generally speaking, this problem can be materially alleviated by greater use of one-way streets, synchronized grid traffic light control, elimination of curb parking and truck loading and unloading during business hours, and all the other regulatory measures that are so politically unpopular.

Other steps that are unpleasant to many municipalities, but which are frequently tremendously effective in older cities, are to correct the inconsistencies in street layout in the central zone and to establish a symmetrical traffic artery pattern providing reasonably continuous flow. In many old cities most

of the streets go nowhere. Because of a complete lack or an incomprehensiveness of early city planning, almost every street jogs back and forth and terminates at a building, a cemetery, or the water's edge. To reach a destination, traffic must frequently follow a circuitous path involving much unnecessary right and left turning. Each turn delays and slows down traffic flow. Although this condition may be only annoying in a suburb, it becomes of major importance in the congested business districts.

Careful analysis of the street plan of many cities frequently will reveal situations where little-used streets can be cut through existing blocks or buildings so as to join with other little-used streets and thus form a continuous artery or where obsolete indentations of a waterfront can be filled and reclaimed so as to permit the extension and the joining of streets terminating at the water's edge. Such measures often will allow unused streets to become valuable traffic thoroughfares and, as an incidental benefit, will often immeasurably increase land values in deteriorated zones languishing in the backwash of a city's growth.

Relief for Mass Transportation

The obvious and the only genuine solution to the traffic problem occasioned by mass transportation equipment (streetcars, trackless trolleys, buses, etc.) operating almost end to end on the city streets is to remove such equipment from the surface of the streets, at least in the areas where it is a major traffic element. Not only will this measure relieve surface traffic, but it also will greatly expedite the movement of the mass transportation equipment and enhance its value to the patrons. Consequently, no long range plan for traffic correction can be considered adequate unless it will ultimately provide off-the-surface mass transportation—for example, a subway system.

However, the natural outline or geographical shape of most cities in the population brackets from 750,000 to 2,000,000 is such that it would be impossible to build a complete, city-wide conventional subway system that would defray its own cost. Subway construction is extremely expensive, and even to approach a self-liquidating basis each line must serve a tremendous number of patrons. When a city is laid out in a long, narrow rectangle, so that a few lines can serve a large and densely populated area, subways can then at least appear to be a sound financial venture. On the other hand, when a city is fan-shaped, as are most United States cities, with streets radiating in all directions and with none of these streets serving a continuous strip of heavily populated zones, the investment required for subways cannot possibly be economically self-supporting.

Under these conditions, the most that can be justified is a composite system consisting of conventional surface equipment (such as streetcars) in all thinly populated areas; and, in the definitely critical zone, either subsurface tubes for these same streetcars or, as an alternate, a relatively small downtown conventional subway system to which patrons can transfer from the radiating surface lines. The business district subway system is, of course, mandatory when buses or trackless trolleys would be used in lieu of streetcars for surface operation in the outer or relatively thinly populated areas.

Elevated rapid transit systems are also practicable, but past experience with this type of facility has demonstrated many shortcomings. Thus, generally speaking, they can be considered undesirable. Also, because the days of the streetcar appear to be numbered and the bus is becoming more popular, it is probable that the central zone subway system is the logical choice for the average city.

The operation of such a system is obvious. From terminal points located on principal arteries a short distance from the central business district, subway trains run into the central zone where they traverse a common subsurface loop or loops routed strategically under important business streets so as to serve office buildings, shopping districts, commercial areas, theater districts, and the like. Transfer privileges are extended to surface transportation. Passengers from a suburb travel by bus along a principal surface radiating street until the subway terminal at the outer fringe of the central district is reached. At this point the passengers leave the bus, descend to the subway, and continue to their destinations in the downtown area, the buses returning to the suburbs. If the passengers wish to continue diametrically across the city, as a small proportion of them may, they stay on the subway until reaching the opposite side of the central zone and then retransfer to radiating surface buses. Of course, buses and other surface equipment should be almost entirely excluded from the area served by the subways.

Another operating possibility in such a system is the convenience offered those who are forced to use automobiles. Since the subway terminals should be located in the deteriorated zones outside the central district, it should be economically practicable to provide extensive parking lots at such locations. Motorists can then leave their cars at these points and, in a few minutes, reach their destination in the business section via the high-speed subway.

This type of off-the-surface facility lends itself to expansion in keeping with the growth of the city and will in no way hamper the normal expansion of the central district. If modern design criteria in lightweight, high-speed railroad practice are applied to subway equipment, it can be smooth, clean, quiet, and reasonably comfortable. Equitable fares and public pressure, returning to mass transportation after 25 years, can make it so.

At only a fraction of the cost of in-city freeway facilities for local traffic—and that fraction a self-liquidating profit-returning investment instead of a nonrecapturable outlay—mass transportation to the central business district can be immeasurably improved. Under a coordinated attack on the problem, increasing restrictions on parking and operation in the downtown area, coupled with the present exasperating traffic congestion, will open up new sources of mass transportation patronage in the outlying districts. New patronage will justify expanded networks of bus coverage. Increased bus patronage will, in time, justify subway extension. Subway extension, offering still faster transportation to the business section, will open up still more sources of patronage. Thus, the cycle will continue of its own volition.

For some obscure reason, decentralization of cities is viewed with alarm by many city planners and the proponents of expressways argue that one of the principal advantages of this type of facility is that it will reduce the tend-

ency to decentralization. The record does not support such a contention. Timid drivers will not venture onto high-speed vehicular speedways. For example, suburban matrons, obliged to travel by automobile, although they would prefer to shop in the downtown stores will often seek out a community shopping center, rather than brave fast-moving or congested traffic. On the other hand, convenient, comfortable mass transportation will bring them back to the downtown stores. The proof rests in the well-known fact that the cities with the most centralization have the most extensive mass transportation systems; and, on the other hand, cities depending largely on motor cars for transportation are the most decentralized.

Redevelopment of Commuter Service

To reduce still one more traffic type contributing to the "downtown" congestion, another type of mass transportation that can be redeveloped in conjunction with the central zone subway system is the commuter service to outlying communities accessible from existing railroad lines serving the city. Such service has languished in many cities, partly because of the automobile. In most cases, however, the deteriorated service is caused by the inconvenience in getting from railroad stations to office and business districts. As a contact with railroad stations is well-nigh mandatory in any downtown subway system, it would usually be possible to provide subsurface rail interconnections at such points and thus to bring the commuter equipment right into the downtown subway loop. With modern equipment designs, it would be a simple matter to devise lightweight rolling stock with retractable third-rail shoes for conventional subway operation in the city and for diesel-electric operation on the surface rails in the outlying sections. Adequate service of this type, right into the heart of the city, would open up whole new groups of patronage in the one-car subsuburban community class.

Conclusions

From a realistic approach to the functional problems of a city as tempered by an appreciation of the economic condition of such communities, it appears that, although urban expressways seem to be the answer for intercity and cross-the-city vehicular traffic, the logical approach to the purely local traffic problem is first to develop mass transportation to the utmost and then, as a secondary measure, to give consideration to in-city expressways for the convenience of personal, luxury transportation. Expensive as subway construction is, it is still cheaper from an over-all viewpoint because, as the nucleus of a composite mass transportation system, it is concentrated at the hub of the trouble and not spread out along the spokes or the rim of the street pattern. The subway nucleus relieves the traffic precisely where the relief is needed—right in the heart of the business district. It disperses the problem outward instead of compressing it into the dense and already hopelessly overcrowded city core. The use of urban expressways for local traffic appears basically unsound. It will not solve the downtown traffic problem, but will only perpetuate it.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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REPORTS

DESIGN OF AIRPORT TERMINAL FACILITIES

REPORT OF THE SUBCOMMITTEE ON TERMINAL FACILITIES OF THE COMMITTEE ON AIRPORT AND AIRWAYS PLANNING OF THE AIR TRANSPORT DIVISION

PART I. INTRODUCTION

This report is the first of a series of studies on the various aspects of airport planning and design to be presented by subcommittees of the Committee on Airports and Airways Planning of the Air Transport Division. To place this material, dealing with airport terminal facilities, in its proper perspective relative to the whole subject of airport and airways planning, it is necessary that an indication of the scope of the activities of two of the other three groups be presented. The Subcommittee on Airport Planning is to stimulate and develop studies and research in the science of planning airports, both civil and military. The Subcommittee on Airways Planning is to stimulate and develop studies and research in the science of planning airways and their civil engineering features on a regional as well as on a local plane. Although the scope of the activities of these two groups is broad enough to cover airport terminal facilities in addition to other airport planning aspects, from a timing standpoint, it appeared advisable to scrutinize the airport terminal facilities problem through a special-purpose subcommittee.

Airport terminal facilities provide common meeting places for the air and the ground services and, in addition, supply those conveniences and necessities expected by the air traveler and the general public normally attracted to transportation terminals. From the standpoint of construction cost, the terminal facilities represent a minor part of the total cost of an airport, but, if properly conceived, developed, and managed, they can produce the majority of the revenue of the airport. Thus, the planning of these facilities should merit more attention in the future than it has received in the past. Another factor contributing to the importance of these facilities is that the general public gains its impression of the airport from the terminal facilities, and public support is essential for continuing airport development and improvement. Fortunately, there appears to be an increasing appreciation of the contribution the airport terminal facilities make to the operational efficiency and the eco-

NOTE.—Please forward all comments on this report directly to Chairman J. B. Bayard, Jr., Director, Airport Div., Horner & Shifrin, 803 Shell Bldg., St. Louis, Mo., or Smith, Hinchman & Grylls, Inc., 243 W. Congress, Detroit 26, Mich.

nomic success of the airport by those officials responsible for the planning and the development of public airports, and consequently a desire on their part to give to this phase of airport development the thought and the study which it needs.

The architect or the engineer faced with a design problem is ordinarily supplied with the basic design criteria and can limit his activities to providing a practical solution to the particular situation. However, this condition generally does not hold with respect to the design of airport terminal facilities. Sometimes it appears that the community officials or their engineer or their architect do not realize that the fundamental policies and data have not been determined. It appears that the architect or engineer called on to plan airport terminal facilities should be prepared to develop the basic information essential to the planning of economical and effective facilities and to assist in the establishment of policies for their efficient use. The preparation of construction drawings and specifications and the supervision of construction is less than half the job.

Determination of the basic requirements for airport terminal facilities of necessity is based on a variety of factors modified by geography, climate, and urban character, ranging from the operational demands of the aircraft operators to the economic policy of the airport owner. It is the purpose of this report to describe methods of establishing airport terminal facility requirements, the essentials to be considered in planning these facilities to serve air transportation and the interested public, the potentialities of these facilities for the production of revenue, and the general approach to the problem of effectuating the planned program. There does not appear, however, to be a formula, into which can be fitted a few items of easily obtained information, and which can be reduced to produce the plan of terminal facilities.

PART II. ECONOMIC SURVEY AND JUSTIFICATION

Because the airport terminal facilities are inseparable from the complete airport from a planning standpoint, it is necessary to consider the complete airport. Certain requirements for the terminal facilities are derived from the economic policy of the community operating the airport, and others are derived from the character of the area served by the airport. The purpose of an economic survey of a community planning a new airport or contemplating improvements to an existing airport is to establish the ultimate use of the airport and its economic function in the community.

More than 90% of all civil airports have been conceived and brought to their present state of development without a clear picture of these basic requirements. This condition is the result of lack of appreciation on the part of public officials of the impact of airports on community life; it is also the result of collective apathy on the part of architects and engineers in performing the planning function where it is most needed in the first preliminary phase.

Extensive publicity has been given to the development of large airports and their related problems in newspapers, magazines, and aviation and engineering journals. Actually, however, the majority of airport and terminal design problems facing engineers and architects in the next several years will be in-

cidental to facilities serving medium-size and small-size communities. However, regardless of whether the engineer faces the problem of airport facilities for either a region having great population concentrations or a small town, the basic planning considerations have to be established if a satisfactory result is desired.

A wide variety of aviation activities takes place on airports throughout the United States. Each type of aviation activity has its own peculiar requirements for landing and terminal facilities. An appreciation of these special requirements has been recognized in several ways: One of these is indicated by the Civil Aeronautics Administration (CAA) classification of airports for planning purposes on a basis of length of landing area or runway available; another is the CAA establishment of categories of airports planned for use by scheduled air transport operations, in which the runway lengths and design loadings are indicative of the type of service that will be rendered the community served by the airport. These categories are called "feeder line," "local," "express," "de luxe," "international airports of entry," and "international express." A more general grouping of flight activities, and airports to accommodate them, results in the designation of military and naval aviation airports or bases; transport flying, including cargo and feeder line operations; private flying airports; and miscellaneous commerical airports, which handle aircraft sales and service, testing, maintenance and repair, flight instruction, charter services, aircraft rentals, and related activities.

It is apparent that before the extent of construction can be determined the use of the airport must be established. From the architect's and the engineer's standpoint, it means acquiring enough information to know whether the problem posed by the community officials actually is the problem.

In some instances the immediate problem is to set up an airport system, an airport administration, and a method of financing. In others, a single airport previously built is to be expanded and adopted for joint scheduled and nonscheduled operations. There may be an urgent need to separate private flying from commercial flying, or national from international, at the same airport, and to provide individual building areas for each. The need for fundamental information is not confined to projects for new airports. This fundamental information must be acquired also for existing airports faced with the necessity of improvement or expansion.

Regardless of the scope of the immediate problem, the essential items of information are always practically the same. The difference is in the degree of thoroughness that must be given any particular part of the investigation to delineate the problem.

The economic survey must be commissioned by a responsible official and funds provided for its cost. To initiate action, the following steps should be considered by the individual or group sponsoring airport development or improvement in the community:

Establishment of a municipal, county, or regional committee or commission to administer and promote local aviation;

Provision of an operating fund for initial expenses; and

Selection of competent engineering consultants to assist the board or the commission in formulating its policies and to provide it with the fundamental information necessary to establish a program.

The same basic information will be contained in a good survey whether for a regional system or for a single airport and its area, or whether the airport is large or small. How is the area determined? An airport area corresponds to an existing area of retail consumer influence served by adequate transportation and community facilities, and the relative importance of the presence or the lack of these facilities must be evaluated after the survey has been made and the analyses have been completed.

Items that the survey should cover may be outlined as follows:

A. General Factors:

1. Growth Indexes.—

a. Population characteristics of the area, including rates of growth, income data, telephones of all types, postal receipts, power consumption, and any other data which, because of local or regional conditions or circumstances, may indicate abnormal departures from the average; and

b. Industry and commerce, such as the volume of retail trade, and the position of the community in the state or region as a center of industrial or commercial activities. Are there demands of specialized industries or of commercial enterprises whose activities would increase the normal demands of the community for air services? Coupled with this general group of data are the employment statistics which indicate the relative diversity, or lack of it, on which the community depends economically.

2. Land Use Pattern.—Public acceptance of airports as community facilities requiring public support is generally well recognized; but the acceptance of an airport as part of a particular individual's neighborhood is open to serious question. It is important, therefore, that the land use pattern of the community be considered in the event that extensive improvements to an existing facility are planned to change the present purpose of the airport, or when a new airport is to be constructed.

3. Ground Transportation.—Regardless of the size of the airport, it should be accessible by the established transit system of the community, if any, and in any event should be located so as to be convenient to the entire community or at least to the population in its own market area. If the airport is to handle scheduled air transport activities, the volume of traffic on the ground to and from the airport will normally be heavier than if the field is to be used for private flying only. In addition, in regard to airports for scheduled air transport use, the size of the area requiring good highway access is much more extensive than the size of an airport for more limited activities.

B. Aeronautical Factors:

1. Determination of the place of the community in the air transportation picture of the United States and determination of the share of the area;

2. Evaluation of local factors influencing this ratio;

3. A similar evaluation for nonscheduled flying, with allowance for local deviations from the normal pattern; and

4. Local patterns and conditions, including meteorological data (the wind rose and what affects it), topographical data on the site or sites under consideration, air traffic patterns of airways in the locality, and existing airport facilities

C. Economic Factors.—The economic analysis of an airport is supremely important. The old adage "Spend in haste and repent at leisure" is especially apt in referring to those developments which have been, are being, and will be planned without due regard to this phase of planning:

1. Income potential must be balanced against capital costs. Sources of revenue are—

a. Revenue from aeronautical activities, consisting of both scheduled and nonscheduled operations—

(1) Landing fees;
(2) Ramp charges;
(3) Terminal building rentals;
(4) Office space rentals;
(5) Utility charges;
(6) Personnel services;
(7) Ground rentals, including charges for communications, buildings, hangars and shops, tank farms, and the like;
(8) Commissions for items such as bus and taxi privileges; and
(9) Fueling charges or commissions.

b. Revenue from obvious nonaeronautical sources—

(1) Private automobile parking and service;
(2) Food service (airport restaurant, coffee shop, and in-flight meal preparation);
(3) Conveniences (telephone booths, telegraph and cable service, lockers, toilets, newsstands, valet service, barber shops, beauty parlors, shoeshine stands, and similar services); and
(4) Advertising.

c. Revenue from merchandising at the airport. Whenever the public congregates in relatively large numbers, as it does at an airport, there is an implicit opportunity for clever merchandising. The ground transportation requirements for an airport generally are such that one boundary is an important highway. The time-distance location factor is such that the airport should be relatively close to the center of population, yet the site should be large enough to offer building sites for stores and other commercial enterprises. Many localities produce specialities, either agricultural or industrial, the producers of which may desire an outlet on the airport for the sale of these products. On the other hand, the community may serve as a distributing center for a large

surrounding area with the airport as a logical sales point for products in demand in the supporting area.

d. Revenue from entertainment and recreation. Often the natural situation for an airport is in open country rather than in built-up areas, and adjacent areas might be developed for various sports. Even a free park in the immediate vicinity of an airport will increase the revenue of the concessionaires of the airport through added airport visitors. It is probable that in some instances the construction of an airport may present opportunity for the development of a lake or a pond which could be capitalized as a community as well as an airport asset.

The survey, when summarized and analyzed, should—

Indicate the best location for the airport from a use and a revenue standpoint;
Permit the determination of the type and the class of the airport;
Indicate the volume of scheduled and nonscheduled aircraft movements and the probable time periods when different levels of operation will be attained;
Permit the estimation of the peak-hour plane movements, which establish the runway, taxiway, and loading apron requirements;
Permit an estimation of the number of personal aircraft to be based on the airport;
Indicate the relative importance of the facility for aircraft servicing, training, maintenance, and overhaul functions in the locality and its contributory area;
Indicate the number of air line passengers using the airport, if air transport activities are involved; and
Disclose the revenue potentialities of the sites under consideration and the extent of the facilities required to permit their operation.

D. Cost Analysis.—Sufficient engineering data are necessary to develop tentative cost estimates of the facilities needed to meet the requirements established by the previous studies. It is obvious that any proposed construction program has limitations as to its permissible cost, which is not always the lowest cost. The revenue potential should be compared to the capital cost in deciding the advantages of competing sites. This same comparison should be made when considering different schemes for the development of the same site.

PART III. BUILDING AREAS, STRUCTURES, AND FACILITIES

Just as the fundamental survey data provide information from which the airport landing facilities design may be accomplished, so also may the data needed for the planning of the building areas be drawn from the survey. It is essential that the airport landing area facilities and the building areas be correlated during design, as each is dependent on the other for efficient operations. Airport usage, as determined in the survey, provides an indication of the types of building areas that should be planned on each airport. The volume of

activities developed gives an indication of the extent of the services and the facilities that will be necessary in each area. However, these general indications should be supplemented by extensive investigation of each airport to modify the general requirements established by the basic survey.

An airport may have several building areas, or the indicated use may be such that one will be sufficient for the type of operations expected.

A. Types of Building Areas.—A building area is a part of an airport set aside for the development of structures and facilities and the conduct of services necessary for the various operations offered at the airport. Many airports have building areas which embrace a variety of functions and it is possible to have efficient operation on this basis up to a certain level of operations. After this level is reached continuation of all activities in one area becomes inefficient.

It is probable that in many instances airports will have more than one area designated for structures and appurtenances, depending on the operational functions and the usages of the airport. The various building areas can be enumerated as follows:

1. Terminal Area.—

a. Service and accommodations for the traveling public;
b. Handling of mail, express, and freight;
c. Concessions and accommodations for the patronage of the general public; and
d. Accommodations for the operational functions.

2. Itinerant and Personal Fliers Area.—

a. Accommodations for operational functions;
b. Sales and servicing; and
c. Storage and maintenance of aircraft.

3. Maintenance Area.—Structures and facilities necessary for routine inspection and adjustment of aircraft.

4. Overhaul Area.—Structures and facilities necessary for the processing of major repair or replacement of aircraft engines, air frames, and their component parts.

(At some major airports the maintenance and overhaul areas might be combined.)

B. Functional Elements for Design.—Every airport is an individual problem, the solution to which results in the design studies prepared after thorough investigation of the several factors involved:

1. Services to Be Performed.—

a. Personal flying;
b. Itinerant and fixed-base operators; and
c. Scheduled air carriers.

(Frequently personal flying and itinerant operations are conducted by fixed-base operators and accommodations for these operations can be combined into one general building area, but should be separated from scheduled air carrier operations and their maintenance at overhaul areas.)

2. Facilities.—

a. Administrative;
b. Public; and
c. Others.

(The functions of administrative space are usually proportionately greater than those of public space for airports having scheduled operations. The public space must be determined by the analysis of peak-hour loads. The separation of functions in an unsymmetrical layout lends itself to flexibility and economical expansion.)

3. Utilities.—

a. Water. An adequate supply must be provided for fire protection and for future expansion of facilities. Both sewage and water systems may, in many instances, require the approval of the state health board, particularly at the air line airport, which will necessitate the approval of United States Public Health Service for the systems used as an integral part of the air carrier services.

b. Power. In this connection it is essential to provide convenient supply source distribution stations and adequate supply and transformer distribution to serve future needs according to the master plan. Emergency generator equipment is a necessity for communication facilities and landing aid devices.

(These services should be available or should be well under way before construction starts on any structure. Electric power should be so supplied to the entire airport that two primary sources are available. In the event of a failure of the major source, the load should be automatically transferred to the other. The scheme should also include automatic transfer back to the original source after it has been energized for a definite period. In the event the second primary source is not practical, then a local auxiliary power supply should be provided as an emergency stand-by for such vital equipment as air line radio, airport control tower radio, and for the runway lighting system. Power should be brought in and distributed on the airport underground.)

4. Ground Transportation.—

a. Control of vehicular traffic;
b. Service drives and loading platforms;
c. Public and employee parking; and
d. Intra-airport transportation.

(Investigation and coordination with local and public roads administrations are essential so that vehicular traffic control will conform with the highway system. Special attention must be given to this phase of airport development, particularly when it involves large airports. Adequate parking area for airport employees must not be overlooked.)

5. Communications.—

a. Airport control tower or CAA radio and visual signal systems;
b. Communications and weather reporting;
c. Airway traffic control center;
d. Public announcement system and visual announcement equipment; and
e. Telephone and telegraph.

(Telephone service conduit should be underground. Besides regular telephone service, the telephone company circuits are used for private line teletype, CAA-Weather Bureau teletype, airways traffic control telephone, and telegraph, and for circuits to remote air line radio transmitters. A decision must be made as to whether one switchboard will serve the entire airport, or whether telephone switchboards will be established for each air line or tenant. In larger terminals, an intra-airport three-digit dial system is a good solution to the problem of intercommunication. Passenger paging in waiting rooms may be accomplished via telephone equipment, or a separate public address system may be installed. In the larger terminal, central paging facilities with specially trained personnel on duty offer a solution to the problem of making a large number of announcements clearly and uniformly. The inclusion of a visual announcement board will reduce the required number of audible announcements. Electrical ducts and special installation details must be given minute consideration if they are to accommodate rearrangement and expansion of facilities as needed.)

6. Concessions.—Facilities and conveniences deemed necessary to the public in the conduct of an efficient, well-managed airport. These may include restaurants, snack bars, shops, and others outlined in the following "check list."

Check List of Functional Elements

1. *Airport Administration:*

a. Office of Airport Manager.—
   i. Reception office;
   ii. Meeting room; and
   iii. Legal assistant and accounting office.

b. Office of Maintenance Superintendent.—
   i. Engineering offices and drafting room;
   ii. Maintenance shops (including carpentry, paint, plumbing, electrical, radio; and others);
   iii. Stock and storage;
   iv. Janitor lockers and toilets;
   v. Employee toilets, lockers, and first aid;
   vi. Garage and workshop for automotive and field equipment;
   vii. Crash and fire facilities;
   viii. Water supply and distribution;
   ix. Sanitation facilities and sewage disposal;
   x. Heating plant;
   xi. Incinerator plant;

xii. Auxiliary power plant and transformer vault;
xiii. Employee cafeteria;
xiv. Telephone exchange or switchboard; and
xv. Terminal corporation or company offices.

c. Public Space.—

i. Passenger waiting room and lounges;
ii. Toilets;
iii. Spectator area and public concourse;
iv. First aid and nursery rooms;
v. Mail drops;
vi. Porters' service room, toilets, and lockers;
vii. Public address system and visual announcement board; and
viii. Chamber of commerce and exhibit space.

d. Public Concession Space.—

i. Telephone booths and telegraph facilities;
ii. Baggage check room or lockers;
iii. Ground transportation dispatching facilities;
iv. Snack bar, coffee shop, restaurant, and cocktail bar, with kitchen and food service facilities;
v. Private dining and club rooms;
vi. Sales and display counters;
vii. Newsstand and cigar stand, machine vending;
viii. Barber and beauty shops, drugstore, florist, valet, etc.;
ix. Concessions for other public services such as automobile service and storage; recreational facilities, including newsreel theater, hotel, and charter and renting services;
x. Bank exchange; and
xi. Other facilities that will tend to develop a community center.

2. Airport Operations:

a. Space for Scheduled Air Carriers.—

i. Ticketing and counter area;
ii. Check-in of passengers;
iii. Workroom;
iv. Information;
v. Reservation workroom;
vi. Operation and station agents' workroom;
vii. Flight control;
viii. Communications space (radio and teletype);
ix. Station supervisory offices;
x. Supply rooms;
xi. Employee toilets and locker space;
xii. Plane crew ready room; and
xiii. Line crew ready room and apron equipment space.

b. Itinerant and Fixed-Base Operations.—

i. Lounge, ticket counter, and service-sales counter;
ii. Operations office and workroom;
iii. Stock and storage space;
iv. Pilots' briefing room (next to communications if available);
v. Employee toilets, showers, and locker space;
vi. Classrooms or meeting rooms;
vii. Line crew ready room and apron equipment space;
viii. Aircraft services (see "Maintenance"); and
ix. Food services (see "Concessions").

c. Personal Flying.—

i. Club lounge;
ii. Operations office and counter space;
iii. Meeting room;
iv. Toilets, showers, and locker space;
v. Stock and storage space;
vi. Aircraft services (see "Maintenance"); and
vii. Food services (see "Concessions").

d. Federal Agencies.—

i. Airport traffic control tower or CAA (usually a CAA function);
ii. Airport controller's office and storage;
iii. Airport traffic control equipment room and toilets;
iv. Airway communications offices;
v. Communications, teletype room;
vi. Communications, radio room;
vii. Communications maintenance shop and offices;
viii. Engine generator room (unless auxiliary power is provided);
ix. Airway traffic control offices;
x. Traffic control and equipment rooms;
xi. Toilets, lockers, and storage;
xii. United States Weather Bureau offices and storage;
xiii. Weather observation and instrument room (outside access);
xiv. Pilot briefing room;
xv. United States Air Postal Transport Service work area;
xvi. Air line postal assistants and loading docks;
xvii. Post office vault, storage, lockers, and toilets;
xviii. Post office trucking platform; and
xix. Public space and counter.

(The Railway Express Agency (not governmental) usually has air express offices adjacent to post office facilities to facilitate service trucking and dispatching.)

e. Federal Agencies at Airport of Entry.—

i. United States Public Health Service offices;
ii. Quarantine waiting room with toilets;
iii. Detention rooms with lavatory;
iv. United States Immigration Service offices;
v. Interrogation room;
vi. United States Customs Bureau offices;
vii. Baggage inspection room and counter;
viii. Bonded storage area;
ix. United States Department of Agriculture office;
x. Plant quarantine and fumigation room;
xi. Offices of the Bureau of Internal Revenue; and
xii. Toilets and lockers for government personnel.

3. Maintenance and Servicing of Aircraft:

a. Air Line Facilities.—

i. Hangars (combined with offices and shops, line maintenance shops, and major overhaul shops);
ii. Warehouse and storage;
iii. Engine and propeller test shops;
iv. Classrooms and engineering offices;
v. Power plant;
vi. Garage and shop;
vii. Fuel storage area;
viii. Aprons and work areas;
ix. Toilets, showers, and lockers;
x. Cafeteria, etc., and
xi. Others.

b. Maintenance Facilities for Fixed-Base or Nonscheduled Operations.—

i. Hangars (combined with offices and shops, line service and maintenance shops, overhaul and storage shops, and unit-hangar stalls for rental purposes);
ii. Sales and exhibit space;
iii. Manufacturing, research, and development;
iv. Mechanic and student training,
v. Fuel storage and service equipment;
vi. Aprons, work area, and tie-down space;
vii. Charter services, passenger and cargo;
viii. Toilets, showers, and lockers;
ix. Private flying clubs; and
x. Others.

c. Apron Facilities.—

i. Scheme of loading positions to serve passengers and cargo;
ii. Turntables;

iii. Apron equipment storage (dollies, stair equipment, and loading devices);
iv. Line crew shelter or ready room;
v. Apron lighting;
vi. Traffic markings;
vii. Fueling service;
viii. Aircraft servicing devices;
ix. Aircraft tie-down anchors; and
x. Others.

4. Overhaul Area.—It is important to keep the over-all scheme flexible and adaptable to new and improved aircraft operational methods. Structural projections should not infringe upon the apron area, and turning or loading positions should not be static so as to "fix" a plane station.

It is important that planning data and space requirements be obtained from the lessee or user agency—that is, the air lines, operators, CAA, and other government agencies that may occupy space in airport buildings. The operational methods of each user influence the building design, including the apron development and its facilities. Space requirements for public facilities should be designed to handle "peak-hour" periods. Calculations should consider on passengers and off passengers, through passengers, those accompanying or meeting passengers, spectators, and employees. Spectator volume will vary, depending on the concession facilities, the distance from town, and similar factors. An example of the correlation of public facilities is shown by Table 1, prepared by the CAA.

TABLE 1.—MINIMUM SPACE REQUIREMENTS IN TERMINAL BUILDINGS

No. of passengers[a]	Components											
	Lobby and waiting (sq ft)	Ticket counter (lin ft)	Ticket office[b] (sq ft)	Baggage room (sq ft)	Toilet Facilities[c]				Dining room (sq ft)	Snack bar (sq ft)	Telephone booths (No.)	Operations office (sq ft)
					Men		Women					
					Water closet	Lavatory	Water closet	Lavatory				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Per person	20	¼	....	1	1/30	1/30	1/20	1/20	15[d,e]	12[e,f]	1/25	6
0 to 50	1,000	25	160	50	2	2	2	2	150	150	4	300
300	6,000	150	1,050	300	7	7	5	5	900	1,000	12	1,800
600	12,000	300	2,100	600	13	....	10	....	1,800	1,800	24	3,600
900	18,000	450	3,150	900	20	....	15	....	3,000	3,000	36	5,400

[a] During peak hour only. [b] 7 ft deep. [c] Allow 1 water closet and 1 lavatory for each 20 employees. [d] 20%. [e] Reduction factor varies for each airport (number of spectators, type of concessions, quality of service, etc. [f] 25%.

It is important to the sponsor and the architect, to prevent loss of time and expense in the initial stage of development, to define specifically in their "contract agreement" and to work up tentative sketches presenting space and circulation arrangement for the review of sponsor and airport users. These

pencil sketches and studies serve as a basis for discussion in obtaining a workable scheme prior to the preparation of preliminary plans.

Any modifications or changes can then be incorporated into the preparation of preliminary plans for a composite design. The preliminary plans will include location and plot plan, floor plans, elevations, and section. The plot plan must show the entire building area and apron arrangement which is in keeping with the principles of the "master plan." Detailed corrections or changes can then be made a part of a conditional approval. However, the architect should not proceed with the final working drawings and specifications until he has obtained a written order to that effect from the sponsor. It might be well to note here that more detailed information regarding the sponsor's relationship to the architect may be obtained from the local CAA office and that the CAA regional offices have architectural advisory services to assist communities on airport building design problems. These services are of particular importance whenever federal aid is contemplated.

The requirements of the United States Public Health Service must not be overlooked for airports having scheduled interstate operations.

Commentary

All references in the remainder of this section are to the "Check List of Functional Elements."

Airport Administration.—As a general rule, the administrative functions may be accommodated in standard office space in the terminal building, in which case it is desirable to confine these quarters to a minimum. Where extensive administrative offices are necessary, they may be located in conventional office space in a separate office building:

Office of Airport Manager.—In large communities the airport manager may need a staff for the administration and the operation of the airport. In other instances the municipal government may supply, through the various municipal departments, the services that would otherwise require a special staff.

Office of Maintenance Superintendent.—At large airports it will be necessary to provide separate quarters and facilities for the maintenance of the airport. For other airports, depending on the size, number, and kind of building areas at the airport, the office of the maintenance superintendent may be combined and administered by the airport manager.

Public Space.—At some large airports, decentralized public services may be necessary for several reasons—(*a*) to reduce the average distance a passenger must walk to reach such services; (*b*) to make more services available to more passengers (greater coverage of the potential market); and (*c*) to reduce the time interval required to reach and utilize such services.

Public service facilities should be located near the center of the passenger and the cargo handling areas because it should be possible for the passenger or the public to reach any part of the station in approximately the same time interval.

Adequate observation decks and terraces should be provided for the sightseers. In addition, enclosed spectator lounges or decks are desirable and should

be located away from the airline ticketing areas. Circulation to and from the public observation decks should by-pass the air line functional elements. Adjacent and convenient to this circulation should be the various concessions and services depending on public patronage.

Public Concession Space.—Essential services should usually be taken care of in buildings for airport administration or passenger and cargo handling. Food service facilities, of which there are many types from the vending machine to the de luxe dining room, must be selected as to type and then so located as to provide the best service to the greatest number of people at reasonable prices. This method of selection and location also applies to newsstand and cigar stand facilities.

Airports require entertainment facilities scaled to the demand. These facilities, when properly located and managed, can become an important source of revenue. Properly located, such facilities will be available to the greatest number of people without interfering with flying activities or the handling of passengers and cargo.

Additional sources of revenue can be created by establishing services and conveniences for the passengers, the public, and the employees. Their location, in relation to the passenger and the cargo handling area and to other areas on an airport, is most important. On the other hand, these services must not become obstacles to the development of the passenger and the cargo handling areas.

Public facilities, including waiting rooms and car parking areas, should be designed to handle the peak-hour requirements of passenger traffic as determined from the normal scheduled peak hour aircraft movements. As 60% of the peak hour traffic can occur within a 20-min period, the peak-hour space requirements should be established on this basis. Calculations should take into account—(*a*) outbound passengers, (*b*) inbound passengers, (*c*) through and transfer passengers, (*d*) people accompanying or meeting passengers, (*e*) spectators, and (*f*) employees.

Airport Operations:

Space for Scheduled Air Carriers.—Structures for passenger handling consist of the passenger station proper with the necessary passenger services and conveniences and the air line operational areas. It should be noted that emphasis must be placed on the necessity for providing ample and sanitary rest room facilities. In the past such facilities have always been entirely inadequate. The large number of women traveling with small children indicates a need for adequate, if not special, facilities.

Air line facilities. Certain air line operational elements must be present in the passenger station and adjacent to the apron. The extent to which other air line functions will be included in the passenger station will be determined by the operating policy of each air line, the type of operation at the airport, and the availability of other air line space. For example, such air line functions as reservations, commissary, flight control, and the like, may or may not be included in the passenger station at certain points. In all cases

the space requirements and layout for present and future use must be obtained from the user air lines.

Loading apron. The design of the loading apron must be closely integrated with the planning of the passenger station as well as that of the airport. The ground handling and servicing requirements of the various aircraft will be the primary guides to apron planning.

Lighting of the apron areas is very important. The over-all lighting system should provide a minimum of 2-ft-candlelight intensity 100 ft out from the barrier or fence. Great care must be exercised in this lighting to reduce the glare factor affecting incoming pilots of taxiing aircraft.

It is necessary to provide a fence or other barrier separating the active apron areas from public areas. Gate and gate identification should be located conveniently to the aircraft parking position to provide simple and direct access to the aircraft.

Cargo handling. Cargo consists of passengers' baggage, mail, express, freight, and air line company shipments. Inasmuch as air line aircraft are operated as combination passenger and cargo carriers, provision must be made to handle cargo in the passenger station area. Freight, when large enough in volume and when carried exclusively by freight aircraft, can be allocated to a specific freight area separated from the passenger area. The rehandling of cargo should be kept to a minimum. The handling of cargo by mechanical methods must be studied as a local problem to determine if it is practical and economical. In planning cargo handling areas, provision must be made for adequate access drives, truck courts, truck docks, protection from the elements, and adequately sized door openings properly designed and protected.

The problem of passengers or porters carrying baggage into or out of the passenger station can be greatly simplified through the use of electric-eye operated doors. Similar electric-eye installations controlling doors to baggage rooms, cargo buildings, and gates in fences used by cargo trains, jitneys, baggage carts, and the like are also recommended.

The flow of passenger baggage must be closely related to passenger circulation. Baggage should be transported directly, simply, and quickly between baggage counters and the aircraft, with a minimum of conflict with other types of circulation.

At small stations where express activity is at a minimum, no facilities are required other than an area in the individual air line cargo room, convenient to the aircraft, to be used for distribution, sorting, and transfer of express shipments (see also "Federal Agencies").

At large stations, an area must be provided for the sorting and the handling of express by express agency employees. This area should be so located that it is easily available to truck traffic and at the same time readily accessible to the loading or unloading of aircraft. The area of this space is controlled by the express activity and can best be estimated by the appropriate agencies.

Separate facilities, when necessary for handling freight, must be expansible and flexible. Since air freight is handled only on an airport-to-airport basis, adequate facilities must be provided for consignors to deliver their freight and for consignees to call for it.

Itinerant and Fixed-Base Operations.—The facilities for itinerant fliers and fixed-base operators will depend on the conditions at the airport under consideration. However, these operations will be found at most airports. The fixed-base operators in most cases will require hangars, sales rooms and display areas, repair and maintenance shops, and the like. Details of facilities will depend on the extent of the operations. Itinerant fliers' needs can well be handled by the fixed-base operators.

Personal Flying.—The needs of personal fliers of course will vary with the local setup. These activities, however, are found at most airports, both large and small. Where the number of personal fliers is sufficient to warrant the organization of a club, club facilities might be necessary. The tendency is toward adoption of the "T" hangar for individual airplanes. It has been found that fixed-base operators can conveniently finance the installation of "T" hangars and rent them to personal fliers at a nominal annual rental. The details of facilities for personal fliers also will necessarily be adjusted to the needs of individual cases.

Federal Agencies.—

Post Office and Railway Express Agency. At airports where a separate United States Post Office Department is indicated, the facilities required are dependent on the volume of traffic together with provisions for expansion. Sufficient space must be provided for an air postal transport service work area, loading docks, workroom, vault, storage, lockers, and toilets. The Railway Express Agency can conveniently be located in the same building with the post office department. Detailed requirements for these facilities will again depend on the size of the airport. It is not necessary that these facilities be located near the center of the airport operations. The approval of the Second Assistant Postmaster General must be obtained with regard to the air mail field post office requirements and design.

At stations where no air mail field post office is contemplated, mail will be delivered to and received from the air line cargo room. Also, at these stations, mail is transferred directly from one air line to another. Where an air mail field post office exists, or is contemplated, all mail will generally be turned over to the post office before being transferred to another air line.

The possibility of all first-class mail being carried by air and the establishment of air mail parcel post indicate the necessity of planning for future expansion.

Airport traffic control. The airport traffic control tower should have an unobstructed view of the airport—that is, the ends and approaches to all runways and all taxiways and aprons. The control tower need not be on top of the passenger station unless it satisfies all requirements by being there. Even if there is no immediate plan for control tower operation at the airport, consideration must be given to the problems that the location and the construction of a control tower will impose should the tower become necessary. Control towers require special planning to make them efficient. The CAA has accumulated a wealth of experience in this regard and should be consulted on control tower design. The airport traffic control tower should be properly

equipped to allow an operator thereof to control adequately air traffic in the immediate vicinity of the airport.

Air route traffic control is ordinarily CAA operated.

Airway communications. This is another function conducted by the CAA. Communications equipment imposes heavy loads, so floors should be designed accordingly.

United States Weather Bureau. The Weather Bureau briefing office should be readily accessible to aircraft pilots. In the case of large stations, Weather Bureau facilities other than the briefing room may be located elsewhere.

Some Weather Bureau instruments require sodded areas on the ground, or adequate roof deck areas suitably designed to protect personnel without adversely affecting instrument recordings. Roof space for wind recording instruments will be required in all cases. At stations where upper air soundings are taken and where electronic equipment is installed, it is necessary to have ground space and suitable equipment shelter.

Federal Agencies at Airport of Entry.—

International facilities; inbound. Space requirements must be planned in consultation with the air lines and the federal agencies concerned. The layout of the international area should provide facilities for these agencies to perform their scheduled duties individually with a minimum of inconvenience to the passengers. The agencies represented are United States Public Health Service, the Bureau of Immigration, the Bureau of Customs, and the Department of Agriculture. Passengers are segregated by flight until they have passed the public health inspection. Under no circumstances is the public permitted access to the clearance area while passengers are being processed. Likewise, space is required for inbound cargo, which is held in bond until cleared by customs.

International facilities; outbound. International clearance operations for outbound passengers can be conducted from the same facilities which serve domestic air lines when domestic and foreign flights are conducted from the same gates. The outbound passenger must have the necessary passports, visas, health certificates, and the like on arrival at the airport. These are examined by the air line employees at the time of check-in prior to departure. Outbound passengers must likewise present evidence of having complied with the income tax regulations of the United States Bureau of Internal Revenue. All these procedures take place at the air line ticket counter. However, space should be provided for customs to examine papers and manifests for outbound aircraft. Where inbound and outbound areas are in proximity to each other, clearance papers can be signed by the officer on duty in the inbound area. Space also must be provided for an immigration officer to check aliens' papers on departure.

Maintenance and Servicing of Aircraft.—Under this heading come the structures for the servicing, maintenance, repair, storage, or protection of aircraft and related equipment:

Air line facilities. Air line activities which are not directly related to passenger and cargo handling operations or to the repair, servicing, or storage of aircraft may be in separate structures at an airport, in hangar lean-tos, or possibly in the airport administration building, or even in the same structure with passenger and cargo handling facilities. These air line activities are special problems, and the solution will differ with each air line and airport. In large congested developments, it is recommended that these activities be separated from the passenger and cargo handling areas.

Hangar and shop facilities for air line use are primarily air line planning problems. A knowledge of their function and component parts is essential to the layout and the design of adequate and proper site areas for such facilities. Sites which are inadequate in area, lacking in utilities, and poor in shape, or which have inadequate or limited access, rough topography, or undesirable soil characteristics, will not attract air lines to develop facilities.

Line maintenance hangar and shop facilities occur more frequently than major overhaul bases, but on the whole require far less ground area for each individual development. Hangar site areas must provide ample room for aircraft storage outdoors, and must permit expansion if that is necessary.

Station shops at other airports can usually be accommodated in the passenger and the cargo handling building or in a small shop building adjacent to a paved apron area.

Fuel storage area. Aircraft fuel storage areas present a problem in site location. Considerations in the location of storage areas are—(*a*) the method of bulk fuel supply, (*b*) the method of dispensing fuel to the aircraft, and (*c*) the minimizing of storage hazards. Fuels are dispensed to aircraft in several ways—

(1) Truck fill, stand to truck to aircraft;
(2) Apron pit to truck to aircraft;
(3) Apron pit or hydrant direct to aircraft; and
(4) Apron hydrant to fueling jeep to aircraft.

Aircraft fuel dispensing systems in the past have usually been designed to meet existing local conditions, and no over-all policy has been followed.

Maintenance Facilities for Fixed-Base or Nonscheduled Operations.—The facilities required for these operators will depend on local conditions and services. The services might include charter flights, sales and display, sightseeing flights, and the like. The site for structures for these facilities at a multi-use airport should be adjacent to the main terminal and so located as not to interfere with scheduled air line operations.

Apron Facilities.—The required apron facilities at any airport will depend on the nature and the density of the traffic. At some major airports special facilities must be developed for loading positions to serve passengers and cargo, and might include elevated platforms from a two-level terminal—cargo on the lower level and passengers on the upper level. At airports with heavy traffic operating on one level, turntables for aircraft might be found desirable.

Utilities pits can conveniently be located on the apron. Such pits should provide at aircraft parking positions all the services necessary to service the

aircraft completely. Provisions at these pits should include back-lighting of parked airplanes.

Studies should be made in designing apron operations to keep the lateral traffic lines clear at all times immediately adjacent to the terminal building proper. It is desirable that all apron equipment be safely stowed away in locations not interfering with other operations at the terminal.

Where underground fueling of aircraft is undertaken, fuel service connections and fuel lines must be kept well away from the buildings or other structures and separate from all other utilities so that in no case could a leaking fuel line contaminate other areas by seepage of a liquid fuel or of fumes.

Overhaul Area.—An air line major overhaul base will vary in size from a few acres to several hundred acres. The specific requirements can only be determined by consulting the individual air line. In all cases, such large developments as major overhaul bases must be segregated from the passenger and the cargo handling area so that the base will not be restricted or restrictive in its development.

An air line major overhaul base may consist of many structures, such as hangars, shops, warehouses, garage, heating plant, office building, engine test cells, and related areas for aircraft parking, employee car parking, and aircraft fuel tank farm.

PART IV. PROGRAM DEVELOPMENT

A. Coordination.—Ordinarily an airport will accommodate a wide variety of aeronautical and nonaeronautical services. Much of the fundamental data necessary for the development of tentative designs may be obtained from the basic survey. However, the final designs must include the specific solutions necessary to solve the problems of the special types of operations to be conducted in the building areas.

The prospective users of the building areas, structures, and services should be given an opportunity to outline their operating problems and in such a way as to give the architect-engineer a clear understanding. For this purpose and to give the prospective airport tenants reasons for particular design decisions the architect-engineer should bring together, for discussions of their requirements, aims, policies, and activities, representatives of the scheduled air transport companies; the fixed-base operators and personal fliers; the municipal, state, and federal agencies; the concessionaires; the utilities; the communications organizations; and the local newspapers. This coordination will be helpful to the designer and at the same time will make a contribution to good public relations, which are so necessary to a successful airport program.

B. Public Relations and the Dollar Problem.—Public and private airport programs are faced at some point in their development with the problem of raising funds for initial capital expenditures. It is frequently difficult to secure local funds even when federal and state financial assistance can be made available. Close coordination between groups interested in aviation is generally required as part of the necessary public relations program.

Generally speaking, all projects in which the public is interested do not proceed to completion until some measure of public understanding has been

attained. It is the obligation of anyone charged with the responsibility for the planning of facilities for public use which involves the expenditure of public funds to acquaint the people with all phases of those efforts, with attention directed especially to the advantages to be obtained therefrom.

A properly conceived and an effectively implemented public relations program can mean the difference between resistance to or acceptance by the general public of the value and the need for airport or airport terminal facilities, and the consequent lessening of resistance to participation in cost. Such a program would serve to defeat the claims of special or selfish interests as opposed to the general public benefit, and would serve as the instructional medium to provide information regarding all aspects of the effect of an airport or airports as they affect the public good, the public necessity, and the social and the economic life of the community.

Recognition must also be given to the difficulty of attaining general public understanding of planning and engineering and of the end to which that planning is directed. Public relations will help interpret the technical decisions for the public and will capture and hold the interest of the public.

An adequate public educational program continuing after the airport is in operation can contribute to the financial success of the airport. Functioning as a sales tool, public relations may be utilized to increase all airport revenues, both flight and nonflight. Adequate and effective presentation of the air travel story cannot help increase the use of air transportation. The airport use and income from privately-owned and company-owned aircraft may be stimulated and increased to a degree not now measurable from past experiences. Nonflight revenues, on which it appears that airport facilities will have to rely for the major part of their income, may also be stimulated and increased to an hitherto unexperienced level exactly as all business relies on a public relations effort to augment income.

Public disinterest or apathy to most airport programs is a major problem, which the public relations program can do a great deal to overcome. This apparent disinterest stems largely from a lack of understanding of the aims and purposes of such programs and of the possible benefits to be obtained therefrom. Airports can be centers of public interest and use. Support equally as enthusiastic as that accorded such projects as municipal parks, civic centers, and the like can be aroused if the planners of the airports make use of all the sales and promotional possibilities inherent in these projects and recognize that it is not sufficient to state only that the project is for the public good. The public itself must be constantly resold on the advantages which accrue to it.

The municipal officials responsible for the operation and the maintenance of the airport are called on annually to provide funds for the maintenance and the operation of the airport. If extensive capital improvements are planned, additional funds must also be obtained; and, if large amounts are required, it is almost a certainty that the approval of the voting public will be necessary.

Recent experiences have indicated the necessity for furnishing the public with more complete financial information with respect to airport construction projects than has heretofore been the practice. These data should include not

only capital costs of the unit of construction currently contemplated, but also estimates of the total capital cost and an idea of the annual maintenance and the operating expenses, together with the possible revenues, to indicate the possibilities of having the annual tax contributions reduced by augmented airport revenues.

The terminal facilities present about the only possibility of reducing the expenditure of public funds through the use of private capital, either directly or indirectly by selling revenue bonds for financing the revenue producing structures. Obviously, the greater the use of private capital, the more public or general credit funds are available for the construction of essential but non-revenue producing facilities.

The establishment of a well-founded financial program as part of the public relations aspect of providing airport terminal facilities also permits setting up a definite schedule of rentals, fees, and lease conditions for negotiations with the prospective tenants, air line operators, fixed-base operators, personal fliers, federal agencies, state agencies, and concessionaires.

In many communities the public officials are not disposed to take on the diverse problems presented by airport development. In others, there are groups which actively oppose airport construction or improvement.

A well-organized airport program will have a phase designed to find out which of the foregoing conditions may affect its successful prosecution, and a program of public education should be conducted whose primary purpose is to overcome the objections which seem to be strongest. It is suggested, therefore, that an examination be made prior to the establishment of a policy for the conduct of the program of such local conditions as public relations, correlation of aviation groups, development of public interest and support, reasons of opposition, selection of a representative who can act as a guiding influence, selection of a competent architectural and engineering firm of airport consultants to carry out the details of the project, local zoning regulations, real property acquisition, and possible purchase of surplus airport equipment from the War Assets Administration.

Respectfully submitted,

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Subcommittee on Terminal Facilities of
the Committee on Airport and
Airways Planning of the
Air Transport Division

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

EXPERIMENTAL DETERMINATION OF VIBRATION CHARACTERISTICS OF STRUCTURES

Discussion

BY SAMUEL J. LORING

SAMUEL J. LORING,[25] ASSOC. M. ASCE.—The writer wishes to thank all the discussers, who have brought out a number of points of practical interest not treated in the paper.

Mr. Klemin has affirmed several of the advantages of the new method which the writer sought to gain by this development. His statement—"* * * the instrument is so light that it should be possible to produce flexural vibrations without torsional vibration"—is considered by the writer to be a specific, although graphic, statement of the fact that the method has considerable selectivity in its ability to isolate and respond to only a single normal mode at a time.

In connection with the testing of actual structures, Professor Bernhard brings out several important points, which certainly deserve attention. The paper was restricted to the use of the method to determine the vibration characteristics of structures according to the classical linear theory of small vibrations—that is, natural frequencies, damping factors, and normal mode shapes. Since these fundamental characteristics are, according to the classical linear theory, fixed structural properties independent of amplitude or specific excitation, the question of amplitudes and probable service conditions does not arise in the restricted treatment of the paper.

These linear characteristics, although admittedly approximations, are themselves of great value in predicting the dynamic behavior of airplane structures under service conditions and especially in investigating aerodynamic instability or "flutter." They might be expected to have the same value in some degree for other structures. The writer agrees, however, that actual structures may deviate somewhat from the linear theory and that it is desirable to give attention to the amplitudes produced by a measuring instrument.

It should be particularly stressed that the amplitudes induced in a structure by this method need not be "extremely small." In testing the 200-ton bridge mentioned by Professor Bernhard it would be wholly feasible to employ, for

NOTE.—This paper by Samuel J. Loring was published in December, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1948, by Alexander Klemin; June, 1948, by R. K. Bernhard, Carl A. Ludeke, and M. J. Turner; and September, 1948, by F. B. Farquharson, and J. J. Slade, Jr.

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example, a 100-lb test mass and to permit 5-in. amplitudes of this test mass. This procedure would lead to bridge amplitudes at resonance of the order of 1/27 in. Considerably larger amplitudes could be produced by further enlargements in the size of the test mass or its movement. Nonlinearities of the types discussed by both Professor Bernhard and Mr. Slade could be thoroughly studied by varying the induced amplitudes in this manner.

The discussion of amplitudes induced in the structure really includes the question of energy input. It may be remarked, however, that the method is extremely economical of "wasted energy." With the small instrument described, adequate responses have been obtained in airplane structures of 100,000-lb gross weight. Testing similar structures by mechanically forced vibrations requires from 2 hp to 5 hp—although not all this power is realized as structural vibration.

Coupling and beat effects are characterized in the linear theory by two or more natural frequencies lying relatively close together. As noted in Section 3, natural frequencies lying 5% apart or less can readily be isolated and measured. The resulting beat phenomena could be predicted if the modes and the frequencies are known.

Observations on the experimental instrument described in the paper were made entirely by direct visual methods and timed with a stop watch. Although the writer has not as yet made experiments in the direction of recording movements of the test mass, it appears that, with the various readily available recording instruments, this problem should offer no real difficulty.

The observation of Professor Ludeke concerning the analogy between the response characteristic of the test mass at resonance and that of a simple single degree of freedom system with damping is very useful for visualizing the character of motions which must be observed in using the method.

Mr. Turner's remarks are of particular interest since he has had occasion to use the principles set forth in the paper in a practical application. However, his unqualified conclusion—"* * * the more elaborate electronic instrumentation in current use is definitely preferable"—does not seem to follow directly from his previous observations that:

> * * * the analyzer appears to have excellent potentialities for the accurate determination of natural frequencies and structural damping. The simplicity of the instrument and its relatively small size are particularly attractive * * *."

Closely spaced natural frequencies can present unusual difficulties. However, from the remarks in Section 3 and from Example 2, it is noted that resolution of natural frequencies as closely spaced as from 2% to 5% apart is readily accomplished. When natural frequencies are more closely spaced, they might be considered substantially identical. There is a straightforward and general method for detecting the presence of two or more identical natural frequencies at a resonance point by examining the observed effective masses at a number of points on the structure.

Consider, for example, the simple case of the in-plane vibrations of a uniform circular ring. It is known that the ring has two identical natural frequencies corresponding to the two modes shown in Fig. 7(*a*) and 7(*b*). The nodes of one of these modes are at the antinodes of the other. When tested

with an instrument located at any point on the ring, these two modes will combine with proportions that result in an antinode at the instrument location, as shown in Fig. 7(*c*). In the radial direction the instrument would show the same effective mass, equal to one half of the mass of ring, at all points of the ring. This observed effective mass would be one half of the mass of ring because the average square of the displacement amplitudes in the sinusoidal mode shape is one half of the square of the peak value at the instrument.

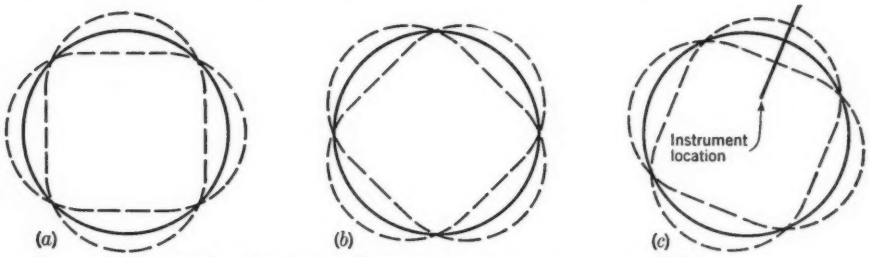

FIG. 7.—MODES OF IN-PLANE VIBRATION OF A CIRCULAR RING

However, if the ring had a single mode with the same effective mass at all points, that effective mass would have to be equal to the whole ring mass not to one half of the ring mass—because a uniform effective mass for a single mode implies a mode with uniform displacement amplitude and all elements of mass of the ring would act with unit effectiveness. It could be deduced from this apparent anomaly in the observed effective masses, therefore, that the ring must have two normal modes of equal frequency at its resonance. These two modes must combine in varying proportions, depending on the instrument location, to produce the observed effective masses.

Such apparent anomalies in observed effective masses will always show up if two or more modes with very closely spaced natural frequencies are being mistaken for a single mode in reading an instrument. If systematic use is made of this fact, no elaborate testing techniques for "resolving" closely spaced frequencies are necessary.

Resonance of the frame of an instrument (not the test mass) must, of course, be avoided. The writer believes, however, that what Mr. Turner has interpreted as the vibration modes of the analyzer itself on a "rigid base" are, in actuality, the response of the analyzer to natural frequencies of Mr. Turner's "rigid base."

In this connection, it is very difficult to find a structure to which the instrument will not respond at resonance. For purposes of calibration, however, a large structure not at resonance is wholly satisfactory.

Professor Farquharson interestingly suggests a specific problem in which the new method may be useful. The solution offered in the final paragraph of Professor Farquharson's discussion is believed by the writer to be a feasible one.

In connection with Mr. Slade's discussion the writer would like to reiterate two points already made: The amplitides excited by the new method need not be "intrinsically" small; and the "lower nonlinearity" mentioned by Mr. Slade could readily by detected or avoided by making tests with two or more induced amplitudes. In this connection, a variation in amplitude can be produced even by a single instrument by varying the amount of initial test mass movement.

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DISCUSSIONS

ELASTIC FOUNDATIONS ANALYZED BY THE METHOD OF REDUNDANT REACTIONS

Discussion

BY ZUSSE LEVINTON

ZUSSE LEVINTON,[34] M. ASCE.—The theoretical basis of the problem of a beam on elastic foundation has been fully developed by the proponents of the various rigorous methods of the theory of elasticity, and the statement of theoretical principles in this paper was necessarily brief. Unfortunately, this brevity has led to some misunderstandings; but, on the other hand, it served to stimulate discussion and round out the subject for the benefit of the interested reader.

Because the writer has introduced a novel method of solution, this part of the paper was treated more fully and has attracted comments on improvements and variations of the method.

Mr. Mensch proposes to improve the method by assuming a parabolic line of deformation. In the case of symmetrical loading with a simple convex or concave "line of deformation," the results will be close to those obtained by the writer's method, as illustrated in Mr. Mensch's example. However, in cases of more complicated deformations with reversed curves or sinuous curves (such as in a "ribbon foundation" with a row of columns), the parabolic assumption would be far from true. The writer's five-panel solution would more closely approximate the conditions.

Mr. Mensch also proposes that a reduction in E be made to account for the flow of concrete in long continued loading. This is justified, provided similar consideration is given to the foundation modulus k. The writer believes that an increase in the effective value of k will compensate to some extent for the decrease in E

Professor Popov points out that the usual theoretical treatments of the problem are based on the Winkler hypothesis, and gives references to the literature on this subject. In 1939, M. I. Gorbunov-Posadov developed the subject along the lines of an elastic semiplane continuum in which the effect of a

NOTE.—This paper by Zusse Levinton was published in December, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1948, by L. J. Mensch, E. P. Popov, Yi-Mai Yao, Jacob Feld, and Victor R. Bergman; and October, 1948, by Joseph Gold, and B. Levine.

[34] Chf. Structural Engr., Knappen Eng. Co., New York, N. Y.

foundation beyond the ends of the beam is taken into consideration. He confined the analysis to two dimensions,[35] and also extended it to three dimensions, to account for the effect of the foundation along the sides of the beam.[36]

The writer's method is based on the Winkler hypothesis, as long as k is considered to be constant along the beam. However, when variation of k along the beam is considered, the method in effect departs from the Winkler method and is an approximation to the case of a continuum. The writer believes that a judicious assumption of a variable k in his method, in effect, accomplishes the same result as do the complicated theories of semiplane and semispace continuums.

By using the term "practical engineer" the writer does not wish to imply that there are two kinds of engineers, "practical" and "theoretical." An engineer becomes "practical" when under pressure of work he is compelled to seek quicker and simpler solutions to his problem; and he uses them, with all due respect for other established methods and theories. The trigonometric series method[12] is one of such practical methods. The writer believes, however, that his method is simpler. Furthermore, the trigonometric series method does not seem to be applicable for the case of variable k.

Mr. Yao defines the limits of applicability of the writer's method and correctly states its advantages. The writer agrees with Mr. Yao that dividing the nonsymmetrical loading into a symmetrical loading and an antisymmetrical loading simplifies the computation. This simplification can be used except in the case of nonsymmetrical variation of k, such as in the vertical pile problem.

The writer agrees with Mr. Feld's statement that "The results obtained from a mathematical analysis are no more accurate than the assumptions used as the basis of the formulas." This principle should be extended to the conventional methods of analysis, as well as to new ones. Of the conventional methods, one is based on the fiction of perfect rigidity of the beam and the other regards the structure as an inverted frame, and the physical properties of the soil are completely disregarded.

The writer's brevity regarding "tension" in the foundation and its "modulus in tension" is probably responsible for Mr. Feld's erroneous interpretation of tension as implying adhesion or "tackiness." In the writer's method, as well as in the "rigorous" methods, based on the theory of elasticity, "tension" of the foundation is regarded differently. It is implied that upward deflections have the same modulus as downward deflections. In properly designed flexible footings, the soil settlement that results from direct compression is greater than the relieving effect of bending, and the ends of the beam do not tend to rise and "pull" on the soil. It is not important, then, whether the soil has, or has not, adhesion or "tackiness" since this property is not mobilized. The condition is similar to the case of loading a concrete block with an eccentric load. If the stress due to direct compression is greater than the extreme fiber stress due to bending, the tensile strength of the concrete is not mobilized and it is imma-

[35] "Tables for Beams on Elastic Foundation," by M. I. Gorbunov-Posadov, Gosstroisdat, Moscow, 1939.

[36] "Calculation of an Infinite Beam Resting on Elastic Semi-Space," by M. I. Gorbunov-Posadov, *Engineering Review*, U.S.S.R. Academy of Science, Vol. III, No. 2, 1947.

[12] "Beams on Elastic Foundations," by M. Hetenyi, Univ. of Michigan Press, Ann Arbor, Mich., 1946, p. 69.

terial whether the concrete has it at all. In computing the extreme fiber stress the modulus in tension is assumed to be equal to the modulus in compression. A case in which "tension" is expressed as resistance to upward pull on friction piles is mentioned in the paper.

In the case of vertical sheet piling, the assumption of "tension" simply means that resistance to soil compression to the left of the piling (for negative deflections) is equal to that to the right of the piling, with the additional assumption that the modulus on either side varies with the depth.

Mr. Feld interprets Fig. 8 as showing results contrary to statics. The writer did not intend Fig. 8 to show any results, it merely indicates in which direction the values of p are to be considered as positive in setting up the equa-

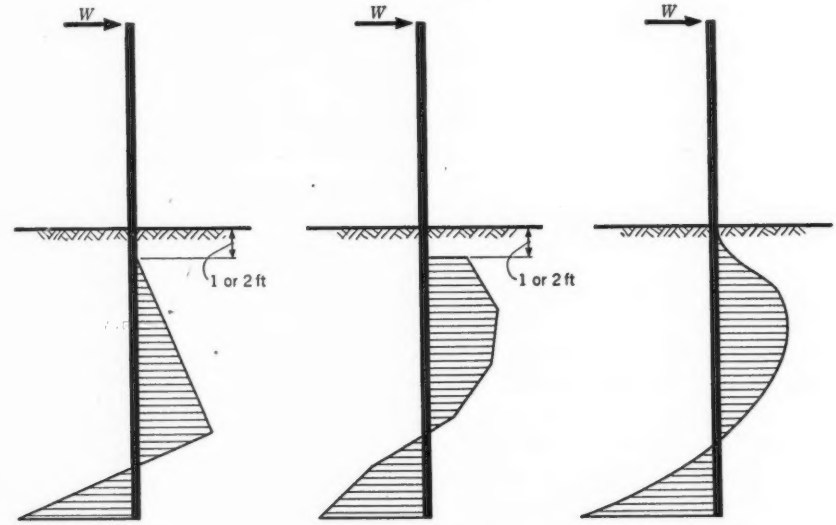

FIG. 20.—COMPARISON OF PRESSURE DIAGRAMS, VERTICAL PILE WITH LATERAL LOADING

tions. When the results of an actual problem are discovered, the reactions at the bottom of the pile prove to be negative—that is, in the same direction as the resultant lateral load, as required by statics. In case of excessive depth of embedment, the pressure diagram crosses the vertical line twice and the reaction at the bottom becomes positive—results which are similar to those obtained by conventional methods of analysis.

The writer agrees with Mr. Feld that the soil reaction at the surface of embedded piling should be zero. To accomplish this, the virtual surface of the "equivalent beam" in Fig. 8 is to be taken 1 ft or 2 ft below the surface. Fig. 20 shows a comparison of pressure diagrams obtained by the conventional method (Fig. 20(a)) and by the writer's method (Fig. 20(b)), as well as what is likely to be the true picture of soil reactions (Fig. 20(c)).

Mr. Bergman makes a restatement of the basic assumptions of the method, which avoids confusion and misunderstanding of the term "elastic." This term originally denoted both the ability to recover shape on removal of the load and the ability to deform in proportion to the applied load. When there is danger of confusing the two properties, a distinction in terminology must be made. Thus, a distinction is made between "elastic limit" and "proportional limit" in testing materials. The term "elastic" lends itself as an adjective to any noun, such as "elastic foundation," "elastic medium," etc. On the other hand, the semantics of the English language does not permit one to say "proportional foundation" or "proportional medium." Mr. Bergman avoids adjectives using nouns "elasticity" and "proportionality" instead, and it is to be noted that the term "elasticity" can stand alone, whereas the other requires an entire phrase: "Proportionality between stress and deformation."

In this paper, "elastic foundation" refers to a foundation that has a settlement at any point proportional to the pressure at the point; and it is not implied that the soil must also have the property of complete recovery on removal of the load. The constant of proportionality k may be assumed to be the same for every point, or it may be assumed to vary along the beam.

Mr. Bergman's references to authoritative statements from the literature on elastic foundations and soil mechanics[21,22] seem to confirm the writer's assertion that an approximate method based on simplified assumptions is desirable and useful.

Mr. Gold considers that the use of the simple beam deflections (y_b) in the absolute terms of the equations is unnecessarily complicated and may be a deterrent to the wide application of the method. He proceeds to develop similar equations by the calculus of finite differences and introduces the beam deflections in another form (Eqs. 33 to 37). In the final analysis, Mr. Gold's method is another variation of the same method of redundants, with equations for each redundant, together with the equations of statics. The difference is in form only. The writer leaves the beam deflection y_b to the individual computer, to be determined by the method most familiar to him (integration, moment area, or a ready formula from a handbook). Mr. Gold expresses the beam deflections in terms of moments taken at every panel point.

It is difficult to agree with Mr. Gold that leaving the choice to the individual would be a deterrent in the wide application of the method. The writer does not agree with Mr. Gold that the calculus of finite differences is "more direct and elementary" compared with the simple algebraic addition of deflections shown in Fig. 1.

With regard to accuracy of the method, Fig. 7 is intended to illustrate the close agreement with the rigorous method of analysis. By scaling the ordinates in Fig. 7, the percentage differences can be established, if desired. Mr. Gold develops the subject further. The curves in Fig. 18 apply to the case of a single central load on the beam, and Table 8 could be used for this case only. How-

[21] "Strength of Materials," by S. Timoshenko, D. Van Nostrand Co., Inc., New York, N. Y., Pt. II, 1941, p. 401.

[22] "Soil Mechanics in Engineering Practice," by K. Terzaghi and R. B. Peck, John Wiley & Sons, Inc., New York, N. Y., 1948, p. 214.

ever, in the writer's opinion, the general recommendation for an accuracy of ±10% may be misleading. The assumed "accuracy" would apply with respect to "theoretical values," presumably derived by the "rigorous method" of the theory of elasticity, based on the Winkler hypothesis. Other, equally rigorous, methods based on semiplane and semispace continuums give values diverging more than 10% among themselves and from the Winkler hypothesis. Under conditions in which the theoretical yardstick itself is uncertain, it is futile to think about 10% "accuracy."

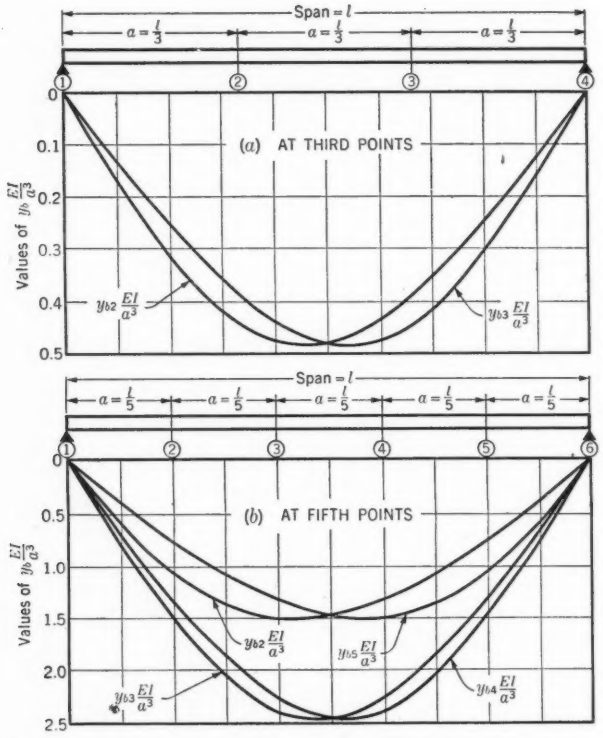

FIG. 21.—INFLUENCE LINES FOR SIMPLE BEAM DEFLECTIONS UNDER A UNIT APPLIED LOAD

The writer recommends that the approximate method of redundants be considered as "accurate" as the "rigorous" methods. When the nature of the loading indicates that the deformed beam has a simple convex or concave curvature, the three-panel solution is sufficient. When a reversal of a curve or a sinuous curve is to be expected, the five-panel solution will give a more desirable approximation. In case of doubt, the five-panel solution should be used; the labor involved is not great in either case.

The ten-panel solution will scarcely find any application in ordinary foundation problems. In the writer's experience, the ten-panel solution was used for long flexible floating dry docks. Under certain conditions of loading the de-

flection curve is sinuous, and with the ten panels it was possible to detect all positive and negative moments along the structure. With water as the foundation material, the theoretical assumptions are fully realized, and the precision of the solution is not illusory.

Mr. Levine discusses the application of the method to a problem with two symmetrical concentrated loads at the ends of the beam. (A number of structures may fall into this group, such as subway inverts, dry docks, locks, reservoirs, etc.) For this loading condition $y_b = 0$. Mr. Levine suggests that for other cases, where y_b must be evaluated, the work could be reduced by the use of influence lines.

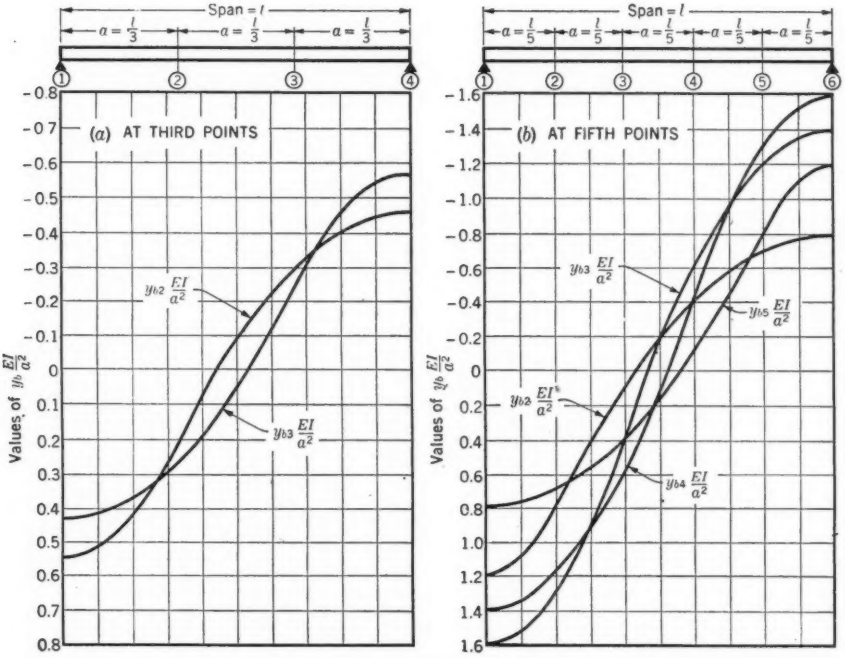

FIG. 22.—INFLUENCE LINES FOR SIMPLE BEAM DEFLECTIONS UNDER A UNIT APPLIED COUPLE

The writer has used influence lines for deflections of simple beams and, in Figs. 21 and 22, has constructed special influence lines that can be used with the three-panel and five-panel solutions. To illustrate their use: In Example 1 (Fig. 3), $m_1 = 15$ ft $= 0.25\,l$. For this position, Fig. 21(*a*) gives $y_{b2} \frac{E\,I}{a^3} = 0.37$ for a unit load, and for the position of the second load ($m_2 = 35$ ft $= 0.583\,l$) $y_{b2} \frac{E\,I}{a^3} = 0.45$. These values must be multiplied each by the load and by $\frac{1{,}080}{a}$; thus: $[(10^k \times 0.370) + (15^k \times 0.45)] \frac{1{,}080}{20} = 564$. For the 30-ft-kip

moment ($m_1 = 0.25\,l$), Fig. 22(a) gives $y_{b2}\,\frac{E\,I}{a^2} = 0.36$. This value is multiplied by M and by $\frac{1{,}080}{a^2}$; thus: $(30 \times 0.36)\,\frac{1{,}080}{20^2} = 29$. Adding the two, $564 + 29 = 593$, which checks with 591.6 obtained by using standard formulas for beam deflections. By a similar procedure the value of $1{,}080\,y_{b3}\,\frac{E\,I}{a^4}$ can be determined by using the curve of $y_{b3}\,\frac{E\,I}{a^3}$ for unit loads, and the curve of $y_{b3}\,\frac{E\,I}{a^2}$ for unit moments.

In conclusion, the writer wishes to thank all participants for their interesting and valuable contributions to the discussion.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ANALYSIS OF FRAMES WITH ELASTIC JOINTS

Discussion

BY RALPH W. STEWART

RALPH W. STEWART,[22] M. ASCE.—The writer is pleased that the paper attracted the attention of exceedingly well-qualified analysts.

Professor Grinter stated that "complexity" is a relative term, and the writer agrees. The intent of the paper was to convey the idea that analyzing structures with yielding joints by the older methods is more complex than analyzing them by the traverse method and also to call attention to the automatic checking feature which Professor Grinter did not mention. Another important and exclusive asset of the traverse method is the direct computation of deflections from the geometry of the traverse as illustrated by Fig. 9 in Mr. Jenkins' discussion.

Professor Rathbun's paper,[23] published in 1935, was the first in known records on the analysis of frames with elastic joints. In the matter of keeping the designer closely in contact with the physical action of the structure as it deforms Professor Rathbun mentions that the other methods "do not have the advantage of giving the analyst the visual concept obtained by the author's diagrammatic method." Those who are interested should compare the last sentence of Professor Grinter's discussion with the first two paragraphs of Mr. Jenkins' discussion and also with Professor Rathbun's comment.

Mr. Lamb's Fig. 11 delineates each item in the deformation of a frame with yielding joints. The writer used this type of illustration in his first investigations of such frames but later combined the Δ-angles with the joint yield and showed ϕ-angles only except at the terminal points of a traverse. Mr. McGee's ingenious device of basing the computations on "yielding-end moments" instead of on fixed-end moments will permit consolidating angles at the terminal points also. This is a valuable variation of the method which, in some cases, will save a substantial amount of computing labor.

NOTE.—This paper by Ralph W. Stewart was published in December, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1948, by L. E. Grinter; June, 1948, by Arthur B. McGee, and C. W. Jenkins; and September, 1948, by George W. Lamb, A. A. Eremin, and J. Charles Rathbun.

[22] Los Angeles, Calif.

[23] "Elastic Properties of Riveted Connections," by J. Charles Rathbun, *Transactions*, ASCE, Vol. 101, 1936, p. 524.

Mr. Lamb's Fig. 10 can be made general by using n to represent the ratio of the joint yield angle to the Δ-angle. The quantity $\Delta/2$ in Fig. 10 will then become $n\ \Delta$, and 1.5 Δ will become $(1 - n)\ \Delta$. Mr. Lamb's solution of a practical design problem indicates how the method can be used to advantage by designers of steel structures who have data on the properties of joints.

Mr. Eremin appends a supplementary graphical construction as an adjunct to the method. Various related types of graphical solutions[24] are used by engineers who learned them as part of their early training. It is questionable whether the use of these methods will grow.

[24] "Moments in Restrained and Continuous Beams by the Method of Conjugate Points," by L. H. Nishkian and D. B. Steinman, *Transactions*, ASCE, Vol. 90, 1927, p. 1.

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DISCUSSIONS

OPERATION OF THE CONOWINGO HYDROELECTRIC PLANT

Discussion

BY ROBERT E. TURNER

ROBERT E. TURNER,[10] M. ASCE.—Mr. Justin has emphasized the point, concurred in by Messrs. Schuleen and Cotton, that the operation of electric generating systems supplied largely by steam plants can be aided greatly by a hydro plant of proper size and character not only because the hydro plant reduces the over-all cost of power generation but also because it supplies additional power capacity on extremely short notice to meet unforeseen load demands, load variations, and forced outages.

In striving for over-all system economy, the sacrifice of most efficient operation of the hydroelectric plant can result in relatively large steam electric operating savings. Thus, the energy generation of the hydro plant is reduced, but the value of the energy generated is increased proportionately. With such operation, the utilization factor defined by Mr. Justin (as the ratio of kilowatt-hours actually generated to the kilowatt-hours which theoretically might have been generated) is not a fair measure of performance. Even if the utilization factor is defined as the ratio of the river flow advantageously used to the river flow available for use, the evaluation of the term, "adventageously used," makes it difficult to compare plants on different systems on the basis of this factor.

Mr. Schuleen's statement that the sharply rising fuel costs place new emphasis on the energy value of hydroelectric generation is indisputable. Certainly during war and postwar periods when all available steam electric generating capacity is pressed into service, the best use of the available hydroelectric generation on an over-all basis is even more important and valuable.

Further extension of the river flow forecasting techniques and refinements in the methods as outlined by Mr. Schuleen and Mr. Williams are more important when it is necessary to set up definite flow estimates for specific periods, particularly when these estimates are used by others to obtain operating

NOTE.—This paper by Robert E. Turner was published in November, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1948, by Joel D. Justin, E. T. Schuleen, and Gordon R. Williams; and September, 1948, by John S. Cotton.

[10] Hydrographer, The Susquehanna Elec. Co., Conowingo, Md.

economies. However, when the potentialities of all the phases of the situation can be mutually recognized, discussed, and understood, it is more valuable to have a quick reliable estimate of river flows to obtain the most probable savings available than to take considerably more time to calculate rainfall losses, and to route the probable flows from a greater number of drainage areas—especially if complete information of precipitation is not immediately available.

Mr. Schuleen mentions the comparatively narrow flow range which is most important to forecast accurately for the purpose of obtaining system operating economies from the Conowingo plant in Maryland. The writer believes there is a similar narrow range of river flows which effect the greatest economy possibilities for every hydro plant on its interconnected system. On a potentially rising river it is usually sufficient to make a preliminary forecast as to whether flows will exceed certain values and then to follow up with more accurate estimates after rainfall ceases, headwaters begin to peak, and forecasting becomes progressively simpler and more reliable. When rainfall is predicted, it is well to analyze the possibilities and to make preliminary estimates of how much flow increase may be expected from several assumed values of rainfall.

It is granted that there would be some advantages in making the more detailed calculations mentioned; but more often, with the many indeterminate factors involved, it is as well to estimate the final answer by a short cut method as it is to calculate all the different individual approximations for which, many times, the complete data are not available when needed. Speed is important and is the major advantage of the short cut method.

Mr. Schuleen's comment that the estimated flow plotted in Fig. 4 could have been revised in light of stage data and rainfall distribution is true, and in actual practice this revision was made, but the fact remains that the basic reason for the difference between the actual and the estimated flows was the effect of channel storage. The flow increase came more from the upper parts of the watershed, thereby bringing into effect more channel storage as the crest moved down to the mouth. The reverse effect of depletion of channel storage on the decreasing river flow is apparent, giving a difference in the opposite direction between estimated and actual flows. The actual volume of water checked quite closely with the estimated volume.

Mr. Schuleen's observation that an error in flow estimates results in greater change of pond level at Holtwood (in Pennsylvania) than at Conowingo is true but in actual practice, since the Safe Harbor plant (also in Pennsylvania) operates on the same system, the Safe Harbor pond can be used to minimize this wider variation; so the effect is about equal. The sum of the pondage per foot at both plants is about equal to that at Conowingo (see Table 1).

Part of the criticism Mr. Williams expresses in regard to the simple rainfall-runoff relations used in Fig. 5 is answered by the fact that the data for each 12-hr period are computed separately before being added to the previous period runoff. Antecedent conditions and character of rainfall can be considered.

Distribution graphs were included to illustrate the method but computation sheets obviate the use of the actual graphs. The suggestion of Mr. Williams that three distribution graphs be supplied for the larger areas to fit different types of rainfall concentrations has the major disadvantage that, unless the

observation and communication facilities are greatly expanded and improved, too often information would be lacking to make the proper choice without delay and an improper choice may be worse than the use of average conditions.

The writer believes the difference in the two recession curves (Fig. 7) is due to greater evaporation losses during the summer than during the winter in the main and the tributary channels of the river. Actually during low flows, the lower part of the watershed seems to deliver more than its normally expected share during the greater evaporation season. However, the writer does agree with Mr. Williams that because data, unaffected by interim precipitation, were meager it is best to use only one recession curve since the inherent inaccuracies of the data chosen may account for much of the difference shown and, finally, the actual use of the curves does not justify the refinement.

Mr. Cotton stresses the importance of the "dependable" capacity of the Conowingo hydroelectric plant and states that insufficient data were given in the paper for the determination of this capacity. For purposes of presentation, the writer limited his paper to a discussion of how the most effective operation of the Conowingo plant on the load of the Philadelphia Electric System at the time was obtained. Actually the plant was, and is, operated on a much larger system, including Federal Power Commission areas 5 and 6, in conjunction with the hydroelectric plants on the lower Susquehanna River listed in Table 1, as well as a 40,000-kw seasonal storage hydroelectric plant and smaller nominal run-of-river hydro plants in interconnected areas.

The writer agrees with Mr. Cotton that firm hydro capacity should be defined as related to alternative steam capacity on the system and not as related only to low river flows. Much confusion is introduced in discussions relating to this matter by indiscriminate use of the terms "firm power" and "firm capacity" as synonymous. Actually "power" and "capacity" do not have the same meaning—power is the rate at which energy is supplied and capacity is the maximum ability to supply power.

In addition, the use of the term "dependable" requires definition or qualification. The determination of "dependable" hydro capacity involves many factors; some are limited to individual systems and some are controversial and not considered in the paper. The writer would like to refer those interested in these matters to a paper and its subsequent discussion published elsewhere.[11]

The writer wishes to express his thanks to Messrs. Justin, Schuleen, Williams, and Cotton for bringing out corollary ideas and for assisting in the clarification of parts of the paper.

[11] "Probability Methods Applied to Generating Capacity Problems of a Combined Hydro and Steam System," by E. S. Loane and C. W. Watchorn, *Transactions*, A.I.E.E., Vol. 66, 1947, pp. 1645–1657.

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DISCUSSIONS

PANAMA CANAL—THE SEA-LEVEL PROJECT A SYMPOSIUM

Discussion

BY ROBERT C. SHELDON, F. W. EDWARDS, AND H. R. CEDERGREN

ROBERT C. SHELDON,[110] ASSOC. M. ASCE.—A clear solution of a very complex problem has been presented by the several authors of this Symposium. The writer will discuss some interesting possibilities in connection with the foundations of the proposed Monte Lirio and Trinidad diversion dams and their relation to the proposed sea-level channel. Having had some experience with foundation conditions on the Atlantic side of the Isthmus, while engaged in the construction of a marine railway at Mount Hope during World War II, the writer believes that the material classified as Atlantic muck must be thoroughly studied before being used to support any load, and that these studies must be made in the immediate location of the load.

The spectacular slides in Culebra Cut or in Gaillard Cut so far overshadow the difficulties of construction on the Isthmus that other troubles have been forgotten. In the history of construction on the Isthmus there have been failures or partial failures of construction founded on the Atlantic muck and on the Pacific muck. A number of these failures have occurred on projects that had been adequately investigated by competent engineers.

During the construction of the original Panama Railroad in the period from 1849 to 1860, great difficulties were encountered in crossing a bottomless swamp between Gatun and Tiger Hill. Settlement of the railroad in this region continued for many years, until somewhat stable conditions were reached.

This stable condition remained until the start of construction of the American canal. The new program called for a double track along the line of the original railroad, but, as the fill for this double track was being made, both the new and the old fills settled. Fills, track, rails, as well as dump cars, were all swallowed up in this swamp as if by magic. Piles placed to support temporary

NOTE.—This Symposium was published in April, 1948, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: June, 1948, by Hans Kramer, and Philip G. Nichols; September, 1948, by George B. Pillsbury, Kenneth S. M. Davidson, Joel D. Justin, W. H. McAlpine, W. E. R. Covell, William Herbert Hobbs, Hibbert Hill, Kenneth C. Reynolds, Gregory P. Tschebotarioff, Charles W. Dohn, and Donald F. Horton; October, 1948, by E. Montford Fucik, Charles M. Romanowitz, and Raphael G. Kazmann; and November, 1948, by George B. Massey, William Allan, and Boris A. Bakhmeteff.

[110] Civ. Engr., Marietta, Ga.

track disappeared, to reappear in other parts of the swamp. With all the waste material then available, the double track was never completed; traffic was, however, maintained on the old single track.

During the canal construction period it was necessary to build the so-called Tiger Hill line from a relocated high level line at Gatun to join the original Panama Railroad at Tiger Hill—requiring fills to a level of approximately +50 ft across this same swamp. The fills were designed for side slopes of 1 on 2; but, as construction of the fill approached finished grade, settlements of 15 ft to 20 ft occurred overnight.

As the existing Panama Railroad embankments to El. +92 were being made, settlements were encountered in the crossings of the Gatun, Buebrancha, Brazos and Baja rivers. At all four crossings the fill was made in successive lifts of 30 ft, 45 ft and 15 ft. The settlement, which amounted to as much as 30 ft, occurred as the last lift was nearing completion. It was necessary to widen the base to prevent uplift of the swamp at the edges of the fill. Final embankment slopes were 5 on 1 rather than 2 on 1 as designed and as shown in Fig. 24 (*a*).

Throughout the records[111] of all these railroad fill settlements the same typical story is told: As the fill was nearing completion everything was in order as the work was left at night—in the morning everything was gone. All these settlements were accompanied by an uplift of the muck outside the fill area.

At the location of the marine railways at Mount Hope a very thorough investigation was made, consisting of closely spaced drill holes and two cofferdam pits in an area 1,200 ft by 300 ft. This investigation disclosed the fact that the top of the sound material underlying the soft, plastic overburden varied abruptly in elevation. Slopes of the sound rock surface running as high as of 1 on 1. In this small area differences in elevation of the sound rock surface were as great as 100 ft, although surface elevations were practically level.

It is the writer's opinion that most of the settlement that has occurred in the fills placed on Atlantic muck are caused by the material sliding on these slopes, rather than by failure in the muck. Also, this Atlantic muck seems to be present at great depths (approximately −200 ft) throughout the area covered by the northern part of Gatun Lake, and the steep slopes of the underlying sound rock found at Mount Hope seem characteristic of this area.

The location of the proposed channel (from Fig. 8) will cross the Gatun River Valley parallel to and about 1,500 ft east of the locations of the original Panama Railroad and the Tiger Hill line, and about 3 miles west of the existing railroad crossing. The location of the west diversion dam is approximately 4,000 ft west of the proposed sea-level channel. The complete section of the proposed construction is shown in Fig. 114.

With the construction program as proposed for the sea-level project, there is little prospect of encountering serious difficulties during the construction of the diversion dams at Monte Lirio and Trinidad. In the dredging of the proposed channel across the Gatun River Valley there is the possibility that considerable excess excavation will be required near the location of the swamp

[111] From "Annual Reports of the President of the Panama Railroad," U. S. Govt. Printing Office, 1903-1913.

encountered by the original Panama Railroad because of the flow of the soft material there into the dredged channel.

It is proposed to empty Gatun Lake in 7 days and to have the sea-level channel in service at the end of the draining period. As the lake is lowered, the pressure that has been holding the uncovered areas of Atlantic muck in a static condition will be released, permitting this material to flow or to slide under the new loads that will have been placed on it in the construction of the spoil banks (El. +65) and the diversion dams (El. +92).

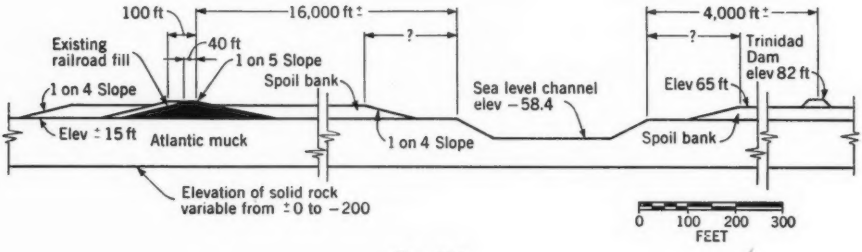

FIG. 114

If these slides occur, they will cause settlement in the spoil areas and uplift of the muck in the areas between the spoil banks and the sea-level channel. If the edges of the spoil bank are near the proposed channel the uplifted muck will flow into the channel, causing a serious block at a critical time.

Although the possibility of a flow of the muck under the Monte Lirio Diversion Dam (with the addition of wide spoil banks to strengthen it), caused by the 55-ft head of the Monte Lirio Lake, is remote, it is a possibility. In view of the importance of this structure to the project and the generous factors of safety planned for other structures proposed, the addition of a sheet pile cutoff wall driven to sound rock at the upstream edge of the existing railroad fill would be warranted.

None of the possibilities mentioned in this discussion constitutes a serious threat to the sea-level project. They can all be avoided by an adequate drilling program aimed at determining the existence of any steep slopes underlying the areas of Atlantic muck at the proposed dams and spoil banks and the proper location of the edges of these spoil banks with respect to any underlying steep slopes. Design of the edge slopes of the spoil bank so that they are very flat (approximately 15 on 1) might be insurance against any settlement.

F. W. EDWARDS,[112] M. ASCE.—This Symposium is complete, interesting, and particularly impressive. Problems unsolved in the past have been studied in detail. Definite values for maximum tidal currents have been determined, and deep dredging has been established as a practical method for developing a sea-level canal. The report that forms the basis for the Symposium reflects the magnitude of work that can be accomplished in a relatively short time by a well-organized and competent engineering staff. Considering the fact that less

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than 2 years elapsed between the date the law was passed (December 28, 1945) and the date the report was submitted to Congress, the amount of data collected and the number of detailed studies completed are enormous. During this period the organization was recruited, facilities were constructed, contracts were let, research studies and preliminary designs were prepared, and, finally, the report was written and assembled. The organization deserves credit not only for the magnitude of the work accomplished but also for the quality of technical thought.

It is interesting under these circumstances to reflect on the relation of these studies to the purpose of the original directive. The following questions appear to cover the essentials of the law passed by Congress—

1. What type of canal would serve best the interests of the United States?
2. Where should such a canal be located?
3. What would be the approximate cost?

The engineering studies reported in the Symposium do not, and could not, provide the basis for the answer to the first question. Whether tidal currents are greater than 4 knots or smaller than 3 knots has little bearing on the answer; nor does it matter how details of flood control are to be handled. Flood control is merely a necessary requirement for a sea-level canal. Reports not available to the general public should furnish some information but the final answer must be based primarily on judgment. The conclusion that a sea-level canal is the only one which might possibly be satisfactory from the standpoint of security was reached on this basis and there is no adequate basis for contradicting this opinion. There is, however, ample reason to believe that a lock canal would not remain usable as a powerful defense facility. It seems, therefore, that the question regarding the type of canal could be answered without such detailed technical studies.

The second and third questions may be considered together as they both involve cost. Relative cost is the primary factor in the second question, whereas estimated actual cost is needed to answer the third question. An examination of Table 1 indicates that the difference in cost between various canal locations is so great that the conclusion that Panama is the most favorable location from a cost standpoint would be reached from less detailed estimates. As evident from the Symposium, only those estimates for locations in the vicinity of the present canal could be considered detailed in any case. Once questions 1 and 2 are answered, question 3 becomes less significant at this time. Would the decision to build, or to reject, the sea-level canal be different if a less detailed estimate indicated the cost to be $3,000,000,000 instead of the reported estimate of $2,483,000,000? It is doubtful, as security appears to be the dominant factor. Congress, which makes the final decision, no doubt will place emphasis on the answers to the first two questions.

The importance of comprehensive studies as detailed as those reported varies with time. When they are made at the beginning of the period of actual design and construction they are invaluable, when made too far ahead of design and construction they are less important. If the decision to proceed with any

of the plans outlined is made promptly, the value of the studies will not be lost. However, if the decision is delayed, the details of the studies will become less valuable as time passes. Practically a year has now (November, 1948) elapsed since the report was submitted to Congress and there appears to be little likelihood that a decision will be reached soon.

The importance of the relationship between time of study and time of decision may be illustrated. As late as 1942 the third locks for the Panama Canal were being designed with lock chambers 140 ft wide and 1,200 ft long with a 45-ft depth over the sill. A short 5 years later the Navy (according to the Symposium) estimated that the ultimate width should be 200 ft, the length 1,500 ft, and the depth 50 ft. A report[113] in the popular press indicates that the latest aircraft carrier is 190 ft wide. By the time the sea-level canal or its substitute is authorized probably many of the detailed studies will be obsolete. This situation should not be created in the case of studies reported to cost $4,500,000.[114] Such detailed studies would be more valuable if they were made immediately before final design and after the basic decisions have been made on the broader questions of what to do and when. Engineers responsible for public works programs would perform a more valuable service if they would insist on this approach in planning projects.

In contrast with the foregoing general discussion the writer would like to direct attention to one specific point, involved in the statement by Mr. Brown (under the heading, "Requirements for Flood Control"), that "the control afforded by Madden Lake has largely ended the likelihood of further major flood disturbances in the present canal."

Engineers in the Panama Canal organization believed for a number of years that Gatun spillway was inadequate. This belief was substantiated by a comprehensive hydrology study covering flood control, water supply, and hydro power development, made in connection with the Third Locks Project in 1940–1941.

This earlier report made use of every available record of rainfall, stream flow, and general storm descriptions. Many old French records that had remained in the vaults for years were transcribed and pertinent data were used in the study. The United States Weather Bureau at that time made an independent study and derived the maximum possible storm over the canal drainage basin. Mr. Brown refers to this storm under the heading, "Spillway Design Flood." It is the storm used to develop the spillway design flood in the present studies. Although the results of the United States Weather Bureau study were received too late to be incorporated in the 1940–1941 hydrology report, the flood-producing characteristics of the storm developed by that agency checked reasonably well with those of the design storm used in that report. The 1940–1941 report emphasized the need for augmenting the spillway capacity for Gatun Lake.

The data available for the studies in 1946–1947 were more complete than those for the 1940–1941 studies. This should be true particularly because

[113] "Armed Forces," *Time*, October 11, 1948, p. 28.

[114] "Can We Defend the Panama Canal?" by Sidney Shalett, *Saturday Evening Post*, October 9, 1948, p. 15.

automatic rainfall stations were installed in 1941 at strategic locations. However, it is doubtful if the recent studies indicate that the Gatun spillway is now adequate. In fact, there is no indication in the paper that the Gatun spillway requirements were studied. This is a problem that should receive further careful study and for which remedial action should be taken.

H. R. CEDERGREN,[115] Assoc. M. ASCE.—The investigation of the effect of transient loads on soil and soft rock (in the eighth Symposium paper) is a valuable contribution in this relatively unexplored field. Unfortunately, much evidence supporting the conclusion of Messrs. Casagrande and Shannon, to the effect that shocks will not endanger otherwise stable slopes in clays and Cucaracha rock, was not included in their paper. An excellent discussion of natural phenomena substantiating this conclusion has been presented by the authors elsewhere.[116] In the Symposium, the authors also point out that loose, cohesionless soils are more susceptible than clays and soft rocks to shear failure under shocks such as might be caused by bomb explosions.

Failures of cohesionless formations under shock are relatively common. Examples of railroad embankments sliding out immediately following the passage of trains are classics in engineering literature. Failures of this type also occurred in the Pacific Northwest during the disastrous 1948 Columbia River flood. A section of a main line railroad embankment on a cohesionless soil foundation slipped into the river taking several cars of a moving freight train with it. At least one major dike failure may have been caused by liquefaction of the foundation that possibly was initiated by the passage of a heavy train over the fill a short time before the failure. Numerous additional examples can be cited as evidence of the potential weakness of cohesionless, saturated soils under shock. The writer knows of no parallel examples for clay soils. Nevertheless, it would seem advisable to collect further factual data regarding large slopes, as recommended by Messrs. Casagrande and Shannon,[116] before ignoring all possible detrimental effects of earthquake or bomb shock on slopes in clays, Cucaracha rock, and other similar materials. A thorough study of records of slides attributed to earthquakes or other shocks would be helpful in this respect. Also, large-scale shock tests of slopes barely stable under static loading conditions should provide valuable information regarding the strength of soils and soft rocks under transient loading.

The authors of the eighth Symposium paper state that the adobe clay subgrade of the Stockton (Calif.) test section (the special test section constructed and tested by the U. S. Corps of Engineers) had a higher modulus of deformation under transient loads than under sustained loads, as typical of clay soils. This observation is consistent with the performance of the test pavements under the heaviest loads ever carried by a single pneumatic tire. For nearly 2 years the writer observed the performance of the specially constructed pavements under both standing and moving wheel loads. Consistently the slowly moving loads (about 5 miles per hr) were less severe on the pavements and caused less

[115] Senior Engr., U. S. Corps of Engrs., Portland, Ore.

[116] "Research on Stress-Deformation and Strength Characteristics of Soils and Soft Rocks Under Transient Loading," by A. Casagrande and W. L. Shannon, *Publication No. 447*, Graduate School of Eng., Harvard Univ., Cambridge, Mass., June, 1948.

deformation in the soil subgrades than standing loads of the same magnitudes. These observations evaluate the combined effect of many factors; however, they are of interest because they reflect the indirect effects of transient and sustained loads on soil masses.

In the seventh Symposium paper Messrs. Binger and Thompson have presented an excellent discussion of the design of cut slopes for the sea-level canal. The design is based on strengths obtained from the analysis of existing Panama Canal slopes supplemented with laboratory tests on representative samples of weaker materials. This approach greatly increases the dependability of new designs. One of the weaknesses of attempting to predict required slopes of excavations from laboratory tests alone is the probable effect of slickensides, faulting, joint planes, and other inconsistencies on the strength of large masses of soil or rock. In the Pacific Northwest, for example, many formations in the hilly regions traversed by highway alinements or proposed relocations of existing highways and railroads around dams contain numerous inconsistencies that are potential sources of trouble from slides. Conditions often vary radically within short distances. In fact, intrusions of weak materials, difficult to detect by field explorations, may cause slides where they are least expected. Because of these conditions, new terrain must be examined closely for indications of old slides, and obviously unstable terrain avoided whenever possible. Careful observation of topography together with well-planned explorations and soil tests can aid in preventing many slides; however, they will not necessarily eliminate all slides. Use of a factor of safety of 1.3 on the Panama project with strengths determined from the study of existing slopes should largely eliminate trouble from slides, as a fairly liberal allowance is made for unknowns. A much larger factor would require considerably more flattening of slopes than proposed. Since the strengths used in the analysis take advantage of extensive experience with the formations in question, the risk of major slides should be slight.

Corrections for *Transactions:* In April, 1948, *Proceedings:* On page 600, line 4, insert "on Cambridge Clay" between "tests" and "were so conducted * * *"; on page 600, line 7, change the sentence beginning "The angles * * *" to "The slopes of the shear planes were carefully * * *"; on page 600, line 10, change "angles" to "slopes"; and, on page 605, line 14, change "10-min" to "5-min." For other corrections see June, 1948, *Proceedings*, page 1021.

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DISCUSSIONS

THE SIGNIFICANCE OF PORE PRESSURE IN HYDRAULIC STRUCTURES

Discussion

BY RAYMOND A. HILL, AND JOHN V. SPIELMAN

RAYMOND A. HILL,[50] M. ASCE.—No process of reasoning can alter the fact that a concrete dam or any other solid body composed of materials heavier than water cannot be supported entirely by water. On the contrary, if the author's plausible reasoning leads to a contrary conclusion, either his premise or his logic must be in error.

For purposes of illustration, a homogeneous triangular dam of the weight and porosity assumed by the author will be considered as being subjected to full buoyancy, "equivalent to a triangular uplift diagram with full water pressure at the back and zero at the face." Such concrete would weigh 145.75 lb per cu ft when dry and 153.25 lb per cu ft when saturated. The total weight of a triangular dam of this density, if saturated, would be $\frac{153.25\ H\ B}{2}$ lb per lin ft, in which H is the height in feet above any substantially horizontal surface and B is the base width at that level. The aggregate of all pore pressures acting upward at the same level (described by the author as a "rough surface joining the lowest pore in each elementary column") would be only $\frac{62.5\ H\ B}{2}$ lb per lin ft. Consequently, even if uplift were to be effective over the "entire gross area of the horizontal section of the dam," only a little more than 40% of the weight of the saturated dam could be supported by the water in the pores of the concrete; the other 60% of the weight would have to be supported by direct contact of solid matter. Incidentally, it can be demonstrated that for any given density and strength of concrete the proportion of the area that must be in direct contact to support the overlying weight is independent of the base width of any triangular dam.

NOTE.—This paper by L. F. Harza was published in the December, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1948, by William P. Creager, J. S. Kendrick, Adolf A. Meyer, E. Montford Fucik, A. C. R. Albery, and W. H. R. Nimmo; September, 1948, by John S. Cotton, Serge Leliavsky Bey, A. H. Davison, R. E. Ballester, John S. McNown, and V. T. Boughton; October, 1948, by Ross M. Riegel, and Douglas McHenry; and November, 1948, by Frederick L. Hotes, R. W. Carlson and Raymond E. Davis, and W. G. Morrison.

[50] Cons. Engr. (Leeds, Hill and Jewett), Los Angeles, Calif.

The same necessity exists for direct contact between solid particles to resist the horizontal load of water against the upstream face of any dam. Such loads can be transmitted to the foundation only by shear, and shearing strength is not one of the attributes of water.

If the author had limited his claims to the potentiality of uplift over most of the area of any section, this could have been accepted because, in the case of dams of ordinary height, built of concrete of customary densities and strengths, there would not have to be as much as 10% of the area of any section in actual contact to transmit all loads to the foundation without failure of the concrete in either compression or shear. However, the author in effect claims that no area of direct contact is necessary when he categorically states that, "nothing can reduce the effective uplift area below 100% whenever pores are filled with water."[λ]

The author conceives of buoyancy as reducing the weight of a body, when as a matter of fact all it does is to reduce the weight to be supported by some other medium than water.

Regardless of the area over which there may be pore pressure, the resultant of all forces above any horizontal section remains the same. The water pressure against the face is not changed; nor is the weight of the dam itself changed, except as such weight is increased by water in the concrete. Whenever water is not present along any section of a dam, the loads are transmitted directly through the particles comprising the concrete and the resultant reaction is equal and opposite to the resultant of all forces above that section. The presence of water under pressure in pores or open seams merely causes some of the load to be transmitted through water from one solid particle of concrete to another; but the total reaction is not altered thereby and the resultant of all forces below the same horizontal section must still be equal and opposite to the unchanged resultant of all forces above that level.

Percolation of water through the pores of concrete can have no adverse effect upon the factor of safety of a dam against overturning, as distinguished from its safety against sliding. On the contrary, a saturated dam would be heavier than a dry dam so that the resultant of all forces above any plane of reference would intersect the base farther upstream. Consequently, the real significance of pore pressures is the extent to which the overlying weight may be supported by water in the pores. To whatever extent this occurs, the resistance of a dam against sliding is reduced because this resistance is proportional to the load supported by direct contact between solid particles.

The concept of internal buoyancy is meaningless and leads to erroneous conclusions. If it were true that the weight of a concrete dam above any horizontal section would be reduced by internal buoyancy, a wet sponge would float.

JOHN V. SPIELMAN,[51] ASSOC. M. ASCE.—This paper has stimulated the writer to give considerable thought to the matter of uplift and buoyancy, leading to the conviction that the reasoning of the author is incorrect with re-

[51] Senior Engr., Design and Constr. of Dams, State Dept. of Public Works, Div. of Water Resources, State of California, Los Angeles, Calif.

spect to a new idea called "internal buoyancy," and that the conclusions drawn therefrom are not proved.

The author attempts to prove that hydrostatic uplift in a dam is effective over exactly 100% of the horizontal area by what he terms a "phase of the buoyancy phenomenon." In the development of his proof the author uses the term, "internal buoyancy" signifying that the concrete surrounding a pore gets a "lift" from the internal pressure in the pore. It seems to the writer that in a material having tensile and shear resistance, as long as the strength is not exceeded, the pore pressure is fully resisted by the stress within the material.

Furthermore, the author, near the beginning of his paper, takes pains to affirm that concrete is porous and that water can seep into all the voids, subjecting all pores to the hydrostatic pressure. There is no need to question this phenomenon as a possibility, but the writer doubts the probability of its occurring in many dams. However, because of the indirect method the author has chosen to prove his thesis, he needs to assume pore pressure throughout the mass; otherwise, by inference, he would not have uplift effective over the full area. The writer wishes to show by buoyancy relations that the opposite is true; that is, the buoyant force on a submerged mass of concrete can only be equivalent to the hydrostatic force on the full area, if the voids are empty. If there is water in the voids the effective uplift area (due to buoyancy) is always less than 100%.

Webster's Dictionary defines "buoyancy" as follows:

> "In physics, the upward pressure by a fluid on a body partly or wholly immersed therein; it is equal to the weight of the floating body, which in turn is equal to the weight of the fluid displaced."

Under the heading, "Buoyancy," the author shows the simple proof for the foregoing definition regarding an impervious body. If the body were porous, with interconnected pores, it would not displace its gross volume but only the volume of impervious matter, assuming, of course, that sufficient time is allowed for the full penetration of the fluid into the pores.

Assume a block of concrete of cross-sectional area A and height d, having a unit dry weight of w_c. Let the unit weight of water be w. Assume, that this block is suspended in a tank of water just to its own depth. If this block were covered with an impervious membrane the buoyancy will be expressed by

$$B = w\,d\,A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39a)$$

As the intensity of water pressure on the bottom of the block is $w\,d$, the effective area of uplift is 100%. This is the condition of buoyancy with the voids empty; but, if the surface of the block is not sealed with a membrane and the water is given time to penetrate into all the pores, then,

$$B = w\,d\,(1 - n)\,A \ldots\ldots\ldots\ldots\ldots\ldots\ldots (39b)$$

in which n is the porosity. Now, because the voids are filled, the effective area of uplift is $(1 - n)\,A$, which is something less than 100%.

Let the block be lowered so that it rests upon the bottom of the tank at the same time lowering the water surface to the top of the block (to simplify the

equations). If some part of the block fits so closely to the bottom as to exclude water the magnitude of the uplift area may be further reduced. However, for the moment, assume that water reaches the entire bottom area as before. Then the buoyancy is unchanged. The block rests on the bottom with a resultant downward force:

$$F = w_c\, d\, A - w\, d\, (1 - n)\, A \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

The second term in Eq. 40 is the uplift and, as before, the unit water pressure is $w\ d$, and the effective area is $(1 - n)\ A$. This would always be less than 100% of the gross area.

The downward force is of greater magnitude than the uplift (which is usually the case in dams). This leads the writer to believe that some impervious areas at the contact would exclude the water, thereby further reducing the uplift as full buoyancy would not be effective.

Mr. Harza gives data on the porosity of concrete which indicate that concrete of good quality has voids aggregating about 12% of the gross volume. The writer has always assumed that by nature the Portland cement and the aggregate are crystalline substances and may be considered impervious. By the foregoing reasoning, concrete having a value of $n = 0.12$ would have an apparent effective uplift area of 88%, assuming full penetration of water in the voids and at least 88% of the bottom area subject to the hydrostatic pressure.

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DISCUSSIONS

REVIEW OF SLOPE PROTECTION METHODS

REPORT OF THE SUBCOMMITTEE ON SLOPE PROTECTION OF THE COMMITTEE ON EARTH DAMS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION

Discussion

BY R. M. MCCRONE, AND HAROLD WEGGEL

R. M. MCCRONE,[38] M. ASCE.—The subcommittee report summarizes information available with respect to various types of protective covers placed on slopes of earth dams and suggests design procedure for riprap slope protection. Also, it discusses briefly the use of bituminous pavements for slopes, questions the feasibility of building a bituminous pavement to withstand severe wave action, and refers to construction of slope protection slabs, using liquid asphalt and graded aggregate to produce a pervious blanket.

Developments since 1935, and especially since 1946, have demonstrated the feasibility of mass placement of properly proportioned hot asphalt mixtures by gravity under water as well as in air, to congeal as amorphous, homogeneous, erosion resistant masses. That the material will withstand serious wave action is evidenced by the asphalt jetty capping and the experimental spur jetty constructed for maintenance of the South Jetty at Galveston, Tex.[39]

Since March, 1947, upward of 200,000 tons of hot asphalt mix (sand asphalt) has been placed underwater for protection of subaqueous banks of the Mississippi River. Placement continued through the high water season, although at some locations depths ranged from shoal water to 90 ft or more, and current velocity often exceeded 12 ft per sec.

Almost all hot asphalt mix placed underwater in the Mississippi River was for blanket revetment of the submerged bank. An exception, at the downstream terminal of the Hardscrabble Bend (Mile 394 above Head of Passes)

NOTE.—This report was published in June, 1948, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: October, 1948, by Howard J. Hansen, William P. Creager, Henry H. Jewell, Joe W. Johnson, and Martin A. Mason.

[38] Prin. Engr., Office of Pres., Mississippi River Comm., Vicksburg, Miss.

[39] "The Design of Asphalt Mixtures for Underwater Construction," by Rossiter M. McCrone and F. C. Field, *Proceedings*, A.S.T.M., Vol. 37, 1937, Pt. II, pp. 499–508.

revetment in Louisiana, is a short wall placed against the bank by use of bottom dump scows to stop continuing raveling of the revetment (Fig. 11). Although the current velocity exceeded 10 ft per sec, the hot mix congealed with a free slope a trifle steeper than 1 on 2.

FIG. 11.—LOW-WATER VIEW OF MASS ASPHALT WALL PLACED FROM BOTTOM DUMP SCOWS DURING HIGH WATER AT LOWER END OF HARDSCRABBLE BEND REVETMENT

HAROLD WEGGEL,[40] ASSOC. M. ASCE.—Formulas for calculating wave heights to be expected under varying conditions are presented in this paper, but no apparent explanation is made for the wide variation in the results shown in Fig. 1. It seems that the degree of conservatism desired is not a satisfactory explanation for changing the coefficient C in the Creager formula (Eq. 5). It is true that the Creager formula can be made to check the Molitor formulas (Eqs. 2), but what explanation is to be given for the Wolf formula (Eq. 4) or the wave heights as determined from the equivalent current velocity formula (Eq. 6) advocated by the Tennessee Valley Authority?

It would be more advantageous to the civil engineering profession if less complicated formulas were put into practice, with a statement as to their limitations as indicated by the data on which they were originally based. Just as the hydraulic engineers have replaced the Ganguillet-Kutter formula by the Manning formula, and are even now replacing the Manning formula as a result of a more complete knowledge of fluid mechanics, so the original Stevenson formula is bound to be replaced for similar reasons. This development cannot occur, however, until sufficient information is obtained to prove or disprove existing formulas.

With $C = 3.41$ the Creager formula closely approximates the Molitor formula; however, no explanation is given for the break in the Molitor formula at a fetch of twenty miles. Perhaps Thomas Stevenson's data, on which D. A. Molitor, M. ASCE, based his formula (Eq. 2*b*), included a number of variables such as slope of bed, depth of channel, and other factors not readily interpretable. If the Molitor formula (Eq. 2*a*) is used for values of D less than 20 miles, and the results are compared with those of the Wolf formula (Eq. 4) the ratio is approximately 1 to 1.82. Therefore, the Wolf formula could be rewritten as

$$h = 1.82 \times 0.17 \sqrt{V D} = 0.310 \sqrt{V D} \ldots\ldots\ldots\ldots (19)$$

[40] Structural and Hydr. Engr., Drury, McNamee & Porter, Ann Arbor, Mich.

Similarly, the Creager formula (Eq. 5) with $C = 2.64$ would be

$$h = (1.6 \text{ to } 1.3)\ 0.17\ \sqrt{V\,D} = (0.27 \text{ to } 0.22)\ \sqrt{V\,D} \quad \ldots\ldots\ldots (20)$$

From a comparison of Eqs. 19 and 20, it is apparent that the maximum correction factor applicable to the simple Molitor formula is less than 2.0, which would apply in the case of breakers. In the case of deep-water waves h is from 1.3 to 1.6 times the normal height, depending on the slope of the beach, the depth of the water, and the angle of approach of the wind producing the wave—factors which as yet have not been fully evaluated for their separate effects.

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DISCUSSIONS

THE RELATIONSHIP BETWEEN PILE FORMULAS AND LOAD TESTS

Discussion

BY W. MACK ANGAS

W. MACK ANGAS,[28] M. ASCE.—An excellent case is made in this valuable paper for the superiority of the Hiley formula over the formulas with which it is compared. It has already been stated that the Engineering-News formula, its variants, and older formulas "die hard." Nevertheless, the engineering profession should recognize and accept the fact that statistical comparisons of the accuracy of older formulas and the Hiley formula clearly indicate the superiority of the latter.

The author is to be congratulated on the manner in which he emphasizes the necessity of limiting the application of dynamic pile-driving formulas to piles driven into noncohesive soils. It is unfortunate, perhaps, that he adds to his warning against the application of dynamic formulas to cohesive soil conditions the statement that by using a formula of the Hiley type it is possible to compute the fiber stress in a pile during driving, even under cohesive soil conditions.

It is doubtful that the stresses in a pile during driving may be determined with reasonable accuracy by the Hiley formula or any of its widely used predecessors, irrespective of the type of soil into which the pile is driven. This doubt is based on pile-driving experiments conducted at San Diego, Calif., early in 1927, the results[18] of which have been referred to by the author. These experiments included the driving and loading of a 20-in. by 20-in. precast reinforced concrete test pile to determine the value of a dynamic pile-driving formula proposed by the Bureau of Yards and Docks, and the subsequent driving of a 20-in. by 20-in. reinforced concrete test pile 65 ft long. The latter pile was not loaded. Two 20-in. by 20-in. Douglas fir test piles and round Douglas fir test piles, were also driven to secure comparative data on the behavior under the hammer of concrete and wood piles. All test piles

NOTE.—This paper by Robert D. Chellis was published in May, 1948, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1948, by Donovan H. Lee; and November, 1948, by Jacob Feld, Richard F. Kafka, G. B. R. Pimm, and George D. Hartley.

[28] Rear-Admiral, CEC, U. S. Navy, New York, N. Y.

[18] "Driving and Loading of Concrete Test Piles at the Naval Supply Depot, Naval Operating Base, San Diego, Calif.," *Bulletin No. 36*, Public Works of the Navy, Washington, D. C., October, 1927.

were driven with a single-acting hammer. In an attempt to determine the stresses occurring in the concrete test pile during driving, a crude homemade strain gage, consisting essentially of a steel tape, three hardwood clamps, and incidentals as later described, was attached to the piles. The lower end of the tape was securely attached to the pile by a hardwood clamp set well above the mud line by a diver. The upper end was attached to a second clamp near the top of the pile by heavy elastic bands cut from an automobile inner tube. The third clamp, to which a small sheet of drawing paper was attached, was then put on the pile near the upper end of the tape. A hole punched in the tape served as a guide for the point of a pencil, which recorded on the drawing paper the deformations in the length of the pile between the point of attachment of the lower end of the tape and the pencil. The device was first used during the driving of the 53-ft concrete pile with a tape 26.8 ft long, the recording paper and the top of the tape being about 11 ft below the top of the pile. The recorded deformation, or the shortening under the hammer blow, was 0.09 in. in 26.8 ft, the corresponding value of F_c being 640 lb per sq in. A repetition of the experiment using a 12-ft tape length, with its upper end as close as possible to the driving cap, yielded a mean deformation of 0.066 in. and a corresponding F_c of 1,054 lb per sq in. This experiment was repeated a third time during the driving of the second concrete test pile, which was 65 ft long. This time a 21-ft tape length was used, the top being again as close as possible to the driving cap. Records were taken from the first stages of driving up to the time the pile was at virtual refusal. The recorded deformations did not increase when increase in resistance developed during driving, but remained practically constant at 0.09 in. in 21 ft throughout the entire driving operation. The mean of nine determinations was 0.093 in., the corresponding value of F_c being 840 lb per sq in.

The method used in these determinations was, of course, crude and subject to error. Nevertheless, the experiment indicates that the stresses produced in a concrete pile by driving are materially higher in the upper part of the pile than in the lower part, and that they are largely independent of the resistance to driving.

In conclusion, the writer believes that there are theoretical flaws in the Hiley formula, and he is also skeptical of the theoretical accuracy of any of the widely used dynamic formulas. Again, this skepticism is based on discrepancies between the observed behavior of piles under the hammer and the behavior predicted by dynamic formulas. A widely used dynamic formula[22] indicates that under identical driving conditions, a reinforced concrete pile should penetrate farther than a wood pile under the force of a given blow, and that, accordingly, a given penetration indicates that a concrete pile will carry a heavier load than a wood pile. Actually, the San Diego experiment showed the reverse to be the case. The penetration per blow of a concrete pile of the same shape and dimensions as a wooden test pile, driven close to it with the same hammer and the same type of cushion, was only a fraction of the penetration of the wood pile when the tips of the piles were at the same elevation.

[22] "Dynamic Pile Driving Formulas," by A. E. Cummings, *Journal*, Boston Soc. of Civ. Engrs., Vol. 27, 1940, No. 1, p. 6.

There is still much to be learned about the mechanics of pile driving, although whether further experimental work can produce dynamic formulas of greater practical value than the Hiley formula is open to question, unless standard forms of cushion can be developed for use with pre-cast reinforced concrete piles. The question of stresses incident to driving is, however, of great importance, and it is believed well worthwhile to consider the experimental determination of these stresses by SR-4 strain gages of the type successfully used in such tasks as the determination of the stresses in airplane parts (even airplane propellers) during flight.

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DISCUSSIONS

VALUATION AND DEPRECIATION OF PUBLIC UTILITY PROPERTY

Discussion

BY DONALD GUNN, HENRY F. LIPPITT, II, AND ALBERT P. LEARNED

DONALD GUNN,[29] ASSOC. M. ASCE.—This timely analysis of recent court decisions treating the subject of depreciation in regulated industry reveals a chaotic situation. It reveals also that, as a result of that situation, this important factor in regulation is rapidly drifting to the point where it will be treated on a purely arbitrary basis, remotely, if at all, related to facts. The writer has recently appeared three times before the Federal Power Commission and twice before the Pennsylvania Public Utility Commission as a witness on the subject, and has clearly observed this trend in full force.

The proposition that determinations of actual depreciation be used to measure the adequacy of depreciation reserves and as a basis for determining a reasonable annual allowance to the reserve was advocated in those cases. In the first two cases, which were before the Federal Power Commission, that proposition was rejected. The decision in the second case before the Federal Power Commission is now on appeal before the United States Court of Appeals for the Third Circuit.

Much confusion in discussions of this subject was caused from the use of loosely defined words such as "inadequacy," "obsolescence," "changes in the art," and the like. The writer will endeavor to avoid increasing this confusion by grouping the various causes of depreciation into the following categories: "Physical depreciation," which includes wear and tear, decay, action of the elements, and so forth; "economic disadvantage," which includes obsolescence, certain types of inadequacy, changes in the art, changes in demand, and the like, that are susceptible of determination through comparative economic studies; and "other causes," which include other types of inadequacy, requirements of public authority, and similar causes not related to economic or physical causes.

NOTE.—This paper by Maurice R. Scharff was published in June, 1948, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1948, by J. Kappeyne; and November, 1948, by W. V. Burnell.

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Mr. Scharff proposes to evaluate depreciation purely through present-worth calculations based on present age and assumed probable life, modified by the present worth of any potential annual savings as measured by comparison with the annual cost of a substitute plant. Thus, when the interest rate and cost estimates are considered factual, as they should be, the sole determinant of the proposed evaluation of depreciation is the assumed probable lives.

The author's proposal appears to be based on the assumption that there is an adequate volume of reliable statistics for various elements of public utility plants. It is generally recognized, however, that in the case of many important elements of property, there are no statistics whatever and there will be none for an indefinitely long time. Even such statistics as are available are of limited usefulness, either because they are not applicable to the type of property now installed, or because it cannot rationally be assumed that the causes of retirements in the past will be repeated in the future. This fact is obvious when the trend in development of steam stations is considered. Twenty-five years ago, approximately 2 lb of coal was required to generate 1 kw-hr of energy, whereas modern steam stations use less than 1 lb per kw-hr. Thus, it is obvious that technical improvement in modern steam stations has reached the point where the margin remaining for improvement, even if perfection were attainable, cannot result in a like degree of obsolescence.

In the absence of adequate and reliable statistics, there is no basis for an estimate of service life except the actual physical and economic condition of the element of property under consideration. If this is determined, however, there is no real need for converting the condition into a service life. The general procedure for making such determination has been outlined elsewhere.[30]

Even if the estimates of time used by Mr. Scharff were construed as "reasonable capitalization periods," they would be of equally doubtful validity. It is unlikely that it would ever be proper to base a selection of property on such long periods of time as those suggested, which, it will be noted, range up to 80 years. Certainly the writer would never recommend a capital expenditure on such a basis. For such purpose, a period of more than 20 years or 30 years would be quite unrealistic, however plausible it might appear as a matter of theory. To the extent it can be appropriately used, it appears more rational to employ this same period of time for the evaluation of depreciation during the life of the property.

Certainly the evaluation of depreciation during the useful life of property must be governed by the same principles which justified its installation and by which its replacement or abandonment will be dictated. When property is new, there is no depreciation; when retirement or abandonment is justified, depreciation in that use is 100%. When property is physically worn out, it is replaced or abandoned wholly without regard to whether it also stands at much or little relative economic disadvantage. In like manner, when economies, available through alternative property, justify replacement or abandonment, physical condition—even though it were near perfect—would have no

[30] "Principles of Depreciation: Final Report of the Special Committee Authorized by the Board of Direction to Analyze and Discuss the 1943 Report of the National Association of Railroad and Utilities Commissioners' Committee on Depreciation," *Transactions*, ASCE, Vol. 111, 1946, pp. 1343–1398.

proper influence on the decision. In practice, however, such cases are seldom met. The more normal situation is where the property is at an economic disadvantage short of justifying replacement, and the total physical depreciation is also short of justifying replacement or abandonment.

In the case of most major elements of property, the physical depreciation is separable into two categories: That which can be eliminated by rebuilding or remodeling, and that which cannot be so eliminated. For example, a generator, turbine, transformer, or building may have suffered physical depreciation of 75%, of which 25% can and must be restored by rebuilding in order to realize further potential usefulness. Obviously, if there is depreciation from no other cause, it will be restored and use will continue. However, if at this time, it stands at an economic disadvantage of 80%, that—together with the imminent 25% rebuilding or renovation cost—will justify replacement. It is important to note that it is only that part of the physical depreciation which is restorable in character that enters into combination with economic disadvantage to justify replacement or abandonment and bring about 100% depreciation.

Thus, the true depreciation in property is either the total physical depreciation, or the physical depreciation that can be eliminated in combination with comparative economic disadvantage, whichever is the greater. This is a fundamental principle that cannot properly be ignored in the evaluation of partial depreciation during the useful life of property. There is no indication that Mr. Scharff has recognized this principle in his proposal. It is not clear how he could have, when he appears to base his treatment on an assumed all-comprehensive remaining life.

Depreciation due to physical causes and that due to economic causes should be treated separately. An evaluation of physical depreciation based solely on an assumed future life is questionable as a practical matter. With the exception of certain mass types of property (poles, crossarms, meters, service connections, and the like) of minor import, there is nothing that resembles statistics which can serve as a guide in estimating remaining life. The more usual situation is where the actual physical condition is the sole indication of remaining life. This being the primary known fact, any assumption of remaining life must be a secondary factor, helpful only in evaluating the economic significance of physical condition. The separate evaluation of physical depreciation affords the opportunity to segregate the restorable from the nonrestorable and thus makes possible the appropriate combination of depreciation due to physical and economic causes.

Depreciation arising out of comparative economic disadvantage should be treated separately from physical and other causes, and on the basis of total annual cost. When property is new, the total annual cost of its use is no greater than that of available alternative property and its depreciation brought about by economic causes is zero. Retirement for economic reasons is justified when the mere out-of-pocket operating cost of existing property is equal to or greater than the operating cost plus all fixed charges (return depreciation, taxes, insurance, etc.) of alternative property. Thus, it seems that the depreciation resulting from comparative economic disadvantage during the useful life of

property can be expressed as a function of these annual costs in terms of capital cost.

The writer does not agree with that part of Mr. Scharff's proposal in which he suggests that depreciation can be translated from a fair value basis to an original cost basis by a simple ratio of reproduction cost to original cost. This proposition seems reasonable where the causes of depreciation are physical, but the writer sees no reason for assuming that it is generally applicable where there is depreciation due to economic causes. Indeed, technological progress is frequently accompanied by an increase of capital cost with a greater-than-offsetting decrease in operating cost. Again, a change in price level is seldom, if ever, reflected correspondingly in both capital and operating cost. In such a situation, it does not appear rational that such a general rule could ever be appropriate.

The author's basic procedure, as well as the suggested simple transition from a fair value basis to an original cost basis, inevitably carries with it a completely erroneous implication—which implication is that because depreciation is properly deductible from reproduction cost in deriving a fair value basis for regulating earnings, it is also properly deductible from original cost for the same purpose. Furthermore, this implication is little weakened by Mr. Scharff's caution to the contrary, and is one of the primary assertions on which zealots in some regulatory jurisdictions seek to confiscate regulated property through manipulation of the straight-line, age-life method of accounting for depreciation.

It can be simply demonstrated and fully proved that rate regulation on the basis of original cost is grounded on completely different principles from regulation based on fair value. For this reason, the writer does not believe that the depreciation problem can be adequately considered from the limited viewpoint of simply measuring and evaluating the effect of the causes of the phenomenon. It is urged, therefore, that the scope of any further consideration of the depreciation problem be broadened to include the proper treatment of depreciation in derivation of rate bases under the various theories of regulation. The writer urges that a comprehensive study of this problem be undertaken forthwith because he is convinced that failure to do so will seriously impair the future usefulness of the Sociey in this field.

HENRY F. LIPPITT, II.[31]—Although Mr. Scharff's paper is a valuable and intelligent theoretical contribution to the field of public utility economics, it is doubtful whether any proposed solution such as this will ever be adopted for public utility rate regulation. For example, Mr. Scharff proposes that reasonable rates should be based on the costs of using the most efficient plant to produce the service. To the extent that costs from the existing plant exceed these "most efficient" costs that difference, capitalized, would represent the measure of accrued depreciation on the existing plant. Thus Mr. Scharff, as an example, shows (see "Closing Comment") that service by a most efficient plant (costing $53,600,000) should cost the consumers $11,777,200 a year. Since the operating expenses of existing facilities are $8,500,000 a year, the difference of $3,277,200 represents the amount available for annual depreciation and return.

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Capitalized at 6%, the present worth of this amount available for annual depreciation and return is $46,969,600, which Mr. Scharff considers the fair value or service value of the existing facilities. Since the existing plant is taken at $60,000,000, the $13,080,400 difference, or 21.7%, is taken as a measure of the accrued depreciation in the existing plant.

Essentially, Mr. Scharff proposes that utility rates be set on a "most efficient" replacement cost, a replacement cost in this instance not only of the existing plant, but a replacement cost of the existing expenses as well. This replacement cost he would use as a "yardstick."

The objections to the use of any such system may be summed up as follows:

1. It is difficult to determine replacement costs of a "most efficient" system, as to required investment as well as to estimated operating expenses;
2. Wide fluctuations would be introduced by such a standard; and
3. There is a serious lack of correspondence between the annual depreciation expense and the indicated accrued depreciation, with resulting unfairness to consumers or investors.

Difficulty of Determining Replacement Costs.—In taking the example which Mr. Scharff used, it was first necessary for him to assume that (*a*) a complete engineering study had been made of the existing plant; that (*b*) estimates had been made of the increase or decrease in cost at the current price level, of the most economical plant, compared to the existing plant; and that (*c*) corresponding estimates were made of the savings in operating expenses.

Plant Costs.—Practically a full page of the paper (see under the heading, "Illustrative Example") expresses the engineering assumptions that would have to be made in computing the cost differences between the theoretical "most efficient" plant and the existing plant (and the corollary savings in operating costs). An indication of the extent of the possible area of difference in the electrical utility field is shown by a "steam station cost survey" which appeared[32] in the technical press on July 3, 1948. The dollars per kilowatt of generating capacity, scheduled for installation in smaller stations, between 1940 and 1945, is shown to vary from at least $100 to $150. At one time a kilowatt of generating station capacity could apparently be set up for as low as $46 (outdoor boiler installation).

The difficulties of determining the replacement cost accurately (which is the very crux of Mr. Scharff's proposal) is adequately illustrated by reference to the Minority Report of the Commissioners of the Commission on the Revision of the Public Service Commission Law in New York,[33] published in 1930:

> "The first objection to the adoption of this standard lies in the extreme difficulty of estimating what it would cost for an entirely different, hypothetical new plant to produce the service. Suppose, for example, that a commission were called upon to fix the charges for electricity in New York City. It would have to determine where the power plants could be most economically located, what kind of generating units would be installed, what transmission and distribution systems would be adopted. Then it

[32] "Fifth Steam Station Cost Survey," *Electrical World*, July 3, 1948, p. 73, Chart 2(*a*).

[33] "Report of the Commission on the Revision of the Public Service Commission Law in New York," *Legislative Document No. 75*, 1930, p. 372.

would have to estimate the capital costs of construction of this entirely new system of generation and distribution. On this estimated capital cost it would have to estimate a rate of return necessary to attract, within some assumed period, the required amount of capital into that new and 'unseasoned' enterprise. It would then need to estimate the probable operating expenses of the system, taking into account on the one hand all the economies that would result from the use of larger generating units, better located power plants, alternating current, 60 cycle equipment (as against the large amount of direct current or 25 cycle equipment now being used by the present company), and taking into account on the other hand the lack of experience of the operating and executive staffs, the large initial expenditures necessary to develop the business and to 'break in' the new equipment. Does anyone really suppose that a Commission could and would really do any such thing? Is it not almost a certainty that the doctrine of 'cost of reproducing the service' would revert eventually to the existing, meaningless doctrine of cost of reproducing a substantially identical plant? We believe that such an outcome is inevitable."

As to the legal aspects of the problem, the leading case applying this so-called "value of service" doctrine is the famous Market Street Railway Case (324 US 548) decided in 1945; but this case is considered an unusually harsh one in utility circles.

Mr. Scharff stated (under the heading, "Illustrative Example") that he

> "* * * has intentionally refrained from discussing the reasonableness of any of the assumptions referred to previously, or any of the numerous interesting collateral subjects that suggest themselves, such as the price level applied in estimates of cost, intangible values (including going concern value)* * *"

and the competition of alternative services. From this statement one can assume that these other problems exist as well.

Operating Costs.—In computing the service value of the service to be rendered, Mr. Scharff makes certain deductions against operating expense to reflect some theoretical savings which might exist by using his "most efficient" plant, the difficulties of determining which are already apparent.

First, it is clear that estimating the annual savings from assumed modernizations will be equally as difficult as estimating the assumed modernizations —if not more difficult. Does the saving in operating expense of $45,000 a year due to the replacement of existing coke ovens depend on (*a*) 100% layoffs of the former employees, (*b*) the calculation of severance pay or pensions for those laid off, (*c*) the existence and use of untrained help in the initial year of the new installation, and (*d*) the effect of possible new wage and "fringe" benefits to the remaining employees? As to material costs, does the annual saving from the assumed modernization take into account the average inventory cost of coal and coke purchased at lower (or higher) prices over the past year against spot prices for substituted oil, natural gas, or liquid propanes—or are prices for the substituted fuels estimated over the coming year? To list such difficulties is only to compound the difficulty of the problem; but an indication of the answer

in the electrical field may be shown by survey data[34] published on July 3, 1948, in which it is stated:

> "* * * a fair average across the country [the United States] shows production costs ranging from as low as 1.5 and as high as 14 mills, with an average value of 4.5 mills per net kilowatt-hour * * *"

Second, it may be seriously doubted whether such assumed savings could be properly deducted according to present court decisions, such as the case of of the Southern Bell Telephone and Telegraph Company versus the Tennessee Railway and Public Utility Commission, decided by the Tennessee Chancery Court on June 17, 1948 (74 PUR NS 150), and later sustained;[35] and, a similar decision by the Georgia Supreme Court.[36] In Mr. Scharff's proposals, to be sure, the excess operating costs of the present plant are not deducted directly from allowed operating expenses, but are related to the present worth of a "most efficient" plant and then deducted; but the result is the same since the total rates of approximately $11,277,200 in Mr. Scharff's example, to be set, in effect disallow some $190,000 of assumed savings.

In other words, perhaps it is difficult enough, both practically and legally, to determine the net rate base to be allowed for rate making purposes by using a theoretical replacement "yardstick" on the invested capital—to say nothing of also applying such a "yardstick" to operating expenses as well.

Wide Fluctuations.—The use of Mr. Scharff's proposed plan would introduce wide swings in the net "service value" or "fair value" rate base to be used as the result of changes in the price level. Today, there are almost no data to show what the yearly variations would be in the cost of replacing a modern gas or electric distribution system efficiently; but data covering the production systems of electric utilities are available, and these show wide fluctuations.

Thus, quoting the cost survey:[34]

> "* * * the plants of less than 60 megawatts built in 1920–1929 and not since expanded cost on the average about $150 per kilowatt. Those built in 1930–1934 averaged about $107 and those in 1935–1939 as low as $76. By 1940–1944 this unit cost had risen to $110 and by 1945–1947 had soared to about $135."

Although these data are for average plants, and an average cost of a "most efficient" plant would be less in each period, the cost of a "most efficient" plant would show similar changes.

The level of operating expenses, including fuel, labor, and maintenance—which are also an essential part of Mr. Scharff's determinations of "fair value" and "service value"—have shown similar and perhaps even larger fluctuations.

The acceptance of Mr. Scharff's proposal would therefore embark the utility business again on the roller coaster of Smyth versus Ames, although this time in the guise of replacement "service value" rather than "fair value" reproduction costs.

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[34] "Fifth Steam Station Cost Survey," *Electrical World*, July 3, 1948, p. 72.
[35] *Public Utilities Fortnightly*, August 26, 1948, p. 324.
[36] *Ibid.*, September 23, 1948, p. 455.

Correlation of Annual and Accrued Depreciation.—As a result of Mr. Scharff's proposal, he arrives at a "service value" for his "most efficient" plant of $46,969,600 compared with an original cost of existing plant of $50,000,000. His annual depreciation expense is approximately $880,600. Although he would retain original cost on the utility's books, he would not apply the annual depreciation expense toward increasing the depreciation reserve to be deducted for rate making purposes. Thus, in spite of the fact that in the 10 years prior to Mr. Scharff's example his consumers might have contributed $8,806,000 in annual depreciation charges reflected in the prevailing level of rates, no equivalent deduction would be made for this contribution. The only measure of accrued depreciation to be deducted from original cost would be the $3,080,000 difference between the "service value" computed by Mr. Scharff's formula and the original cost contributed by the investors in the utility's plant. Therefore, this would ignore, for rate making purposes, more than $5,000,000 contributed by consumers in the preceding 10 years. As Mr. Scharff states (in the paragraph preceding "Closing Comment"):

> "* * * both the regulatory commissions and the public utility industry would be well advised to separate completely the accounting requirements from the methods of estimating service value."

That this treatment runs contrary to a fair division between a public utility's consumers and its stockholders was recognized as far back as 1909 when the United States Supreme Court stated, in ruling on the case of the Louisiana Railroad Commission versus the Cumberland Telephone Company (212 US 414):

> "It was obligatory upon the [utility] to show that no part of the money raised to pay for depreciation was added to capital, upon which a return was to be made to stockholders in the way of dividends for the future. It cannot be left to conjecture, but the burden rests with the [utility] to show it. It certainly was not proper for the [utility] to take the money, or any portion of it, which it received as a result of the rates under which it was operating, and so to use it, or any part of it, as to permit the company to add it to its capital account, upon which it was paying dividends to shareholders. If that were allowable, it would be collecting money to pay for depreciation of the property, and, having collected it, to use it in another way, upon which the [utility] would obtain a return and distribute it to its stockholders. That it was right to raise more money to pay for depreciation than was actually disbursed for the particular year there can be no doubt, for a reserve is necessary in any business of this kind, and so it might accumulate; but to raise more than money enough for the purpose and place the balance to the credit of capital upon which to pay dividends cannot be proper treatment."

Similar judgments were handed down in the following cases: The Lincoln Gas Company versus Lincoln, Nebr. (223 US 349) in 1912, the Clarks Ferry Bridge Company versus the Commission (291 US 241) in 1934, Lindheimer versus the Illinois Bell Telephone Company (292 US 151) in 1934, and the St. Joseph (Mo.) Stockyards Company versus the United States (298 US 38) in 1936.

This correlation between the annual depreciation expense and the accrued depreciation on a utility's books deductible for rate making purposes is followed by perhaps most regulatory commissions today,[37] and is the position adopted by the National Association of Railway and Utilities Commissioners.[38] Mr. Scharff's proposal therefore runs counter to the well-defined trends in present day regulation.

Summary.—Although it may be possible that "the ghost of Smyth versus Ames still walks," it is increasingly clear that his ephemeral peregrinations have become greatly limited within recent years. Mr. Scharff adequately points out the signs of the times in the first few pages and this is reinforced by the comparative results of the Federal Power Commission's recent survey[37] with a similar one in 1941 which indicates that the direction of public utility regulation today is toward the acceptance of original cost less the annual depreciation accrued on the company's books (rather than any engineering estimates of "service value" or "fair value," however precise and accurate such estimates may be).

In the last paragraph of his paper, Mr. Scharff points out that an application of his theory has been accepted as a "fair standard of value" for a condemnation award for certain gas and electric distribution property taken by the federal government. Unfortunately, whatever the merits of Mr. Scharff's proposal may be in valuation for condemnation purposes, they do not appear to be likely of acceptance today in the rate making and regulatory field.

ALBERT P. LEARNED,[39] M. ASCE.—Recent United States Supreme Court decisions are reconciled by the author with the Smyth versus Ames decision,[12] although at the same time attention is called to the different bases used in the decisions. This follows the idea contained in Justice Robert H. Jackson's comment that the federal regulation of the natural gas industry was a performance of economic rather than of legalistic rituals. Furthermore, Mr. Scharff states that the accounting method of providing for depreciation is not a fair measure of the actual depreciation, but may still be a convenient procedure in the accumulation of the reserve.

Apparently, the author is making the points that:

(1) "Fair value" and "service value" are identical;

(2) Service value is measured by the most modern and efficient equipment of capacity to meet the demand;

(3) Accrued depreciation of an existing property is the difference between its "reproduction cost new" and the "service value";

(4) A fair return to the owner, as well as payment by the user, for what the service is reasonably worth cannot be secured by use of either reproduction cost new less depreciation, or original cost less depreciation, where the observed

[37] "State Commission Jurisdiction and Regulation of Electric and Gas Utilities," Federal Power Commission, U. S. Govt. Printing Office, Washington, D. C., 1948, p. 8.

[38] "Report of Committee on Depreciation," National Assn. of R. R. and Utilities Commrs., Washington, D. C., 1943, pp. 154–156.

[39] Asst. Engr., Black & Veatch, Kansas City, Mo.

[12] "Does the Ghost of Smyth v. Ames Still Walk?" by Robert L. Hale, *Harvard Law Review*, Vol. 55, 1942, p. 1116.

or life method is used to determine depreciation, the result being "reasoning in a circle";

(5) Fair value is measured by capitalizing, at the selected rate of return, the difference between the cost of operation of a modern plant plus its required annual depreciation allowance, and the return on the estimated cost of a modern plant less the cost of operation and the annual depreciation allowance on the existing plant; and

(6) The substitution of the modern plant with its costs introduces the element of competition into the solution of the problem.

The writer feels that this procedure could only bring simplification if there were generally accepted bases for both the cost of construction and the operation of a plant proper to replace the present plant. If such measures are in controversy, it would add to the problem. The relative value of proper design might even be injected into the considerations.

If Mr. Scharff's method has been analyzed correctly, the accrued depreciation would be constant if there was no change in the art and there was no change in operating expense caused by lessened efficiency of equipment, costs varying with output, or added maintenance costs. The element of functional depreciation used in reproduction cost new less depreciation appraisals reflects many of the points Mr. Scharff's plan is intended to accomplish. The writer has always proceeded on the basis that obsolescence should be recognized in the observation method. The author's method seems to the writer to be even more complicated than the normal valuation procedure.

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DISCUSSIONS

FLEXURAL CONSTANTS OF HAUNCHED BEAMS BY AREA COMPUTATION

Discussion

BY J. EDMUND FITZGERALD, AND HARRIS SOLMAN

J. EDMUND FITZGERALD,[14] JUN. ASCE.—The author has shown true originality by avoiding the generally accepted methods of approach based on the $\frac{M}{EI}$-diagrams and launching his investigation on the basis of the relative reciprocal I-diagram. This method enables one to construct tables for haunched beams with no-load conditions. By including the values of the momental areas A for various haunches it is then possible to derive expressions for end moments quite directly for any load conditions.

The graphical method of determining A_1, A_2, A_3, etc., suggested by the author proves too involved with respect to the number of lines required and the necessity of multiplying all values by L, to compete favorably with the analytical method. The writer solved a number of haunches by both methods and is decidedly in favor of the analytical method.

The most encouraging aspect of this paper is the fact that one may now approach the problem of haunched beams with far more freedom than has ever before been possible. Beam tables, at best, have been only an expedient made necessary by the formidable amount of calculations heretofore required in the solutions of haunched beams. The author's method is rapid and adaptable to "mass production" methods of calculation. The use of the Simpson rule to calculate areas is no more accurate than when used to calculate the areas of $\frac{M}{EI}$-diagrams, but the calculations of ordinates have been changed from a second source of error to an exact method in so far as each ordinate is concerned.

The theorem of similtude is by far one of the most valuable contributions made by the author.

NOTE.—This paper by Fang-Yin Tsai was published in September, 1948, *Proceedings*.

[14] Asst. Prof. of Civ. Eng., School of Eng., North Dakota Agri. College, Fargo, N. Dak.

In Eqs. 44 to 49, inclusive, the author avoids the direct approach for no apparent reason. Actually, his analysis mars the structural concept. The more generally used and understood approach is as follows:

Referring to Fig. 5,

$$M_{ok} = [\tfrac{1}{2} k L x (2 - k) - \tfrac{1}{2} x^2] w \ldots\ldots\ldots\ldots (141a)$$

and

$$M_{kn} = \frac{k^2 L}{2} (L - x) w \ldots\ldots\ldots\ldots\ldots (141b)$$

From Eq. 26,

$$E I_o (\alpha^o{}_{n(3)} + \alpha^o{}_{o(3)}) = \int_o^n M y \, dx = \left[\tfrac{1}{2} k L (2 - k) \int_o^k y x \, dx - \tfrac{1}{2} \int_o^k y x^2 \, dx + \frac{k^2 L^2}{2} \int_k^n y \, dx - \frac{k^2 L}{2} \int_k^n y x \, dx \right] w$$

$$= \frac{w}{2} [k^2 L^2 A_{(kn)} + k L (2 - k) A_{1(ok)} - k^2 L A_{1(nk)} - A_{2(ok)}] \ldots (142)$$

From Eq. 27,

$$E I_o \alpha^o{}_{n(3)} = w \left[\tfrac{1}{2} k (2 - k) \int_o^k y x^2 \, dx - \frac{1}{2L} \int_o^k y x^3 \, dx + \frac{k^2 L^2}{2} \int_k^n y x \, dx - \frac{k^2 L}{2} \int_k^n y x \, dx \right]$$

$$= \frac{w}{2} \left[k^2 L A_{1(kn)} + k (2 - k) A_{2(ok)} - k^2 A_{2(kn)} - \frac{1}{L} A_{3(ok)} \right] \ldots (143)$$

Subtracting Eq. 143 from Eq. 142,

$$E I \alpha^o{}_{o(3)} = \frac{w}{2} \left[k^2 L^2 A_{(kn)} + (2 + k) k L A_{1(ok)} - 2 k^2 L A_1 - (1 + 2 k) A_{2(ok)} + k^2 A_2 + \frac{1}{L} A_{3(ok)} \right] \ldots\ldots\ldots\ldots (144)$$

Eqs. 143 and 144 agree with the author's results, but are found in a more direct manner.

HARRIS SOLMAN,[15] M. ASCE.—The avowed purpose of the author, as stated in the "Synopsis," is to present a method of expressing the flexural constants of a beam in terms of the characteristics of the beam as represented by the momental areas of the reciprocal *I*-diagram and in the form of exact, easily evaluated formulas. The use of this method is not limited to any particular type of beam and may be applied with equal advantage to cases of beams of any desired sectional variation and even to prismatic beams. It is

[15] Highway Bridge Engr., U. S. Public Roads Administration, Div. 1, Albany, N. Y.

difficult to understand, therefore, why the author chose to present his discussion under the title and within the framework of "haunched beams," which may be mistakenly assumed to imply a restriction of the discussion to one particular type of beam.

Of especial interest to the writer is the type of beam in which the section undergoes abrupt changes at a number of points in the span—the points dividing the beam into a number of segments of cross sections different from each other but constant for the length of each segment. This is the beam of prismatic segments and it represents the type to which belong the most steel beams where the sections are built up by the addition of cover plates. In spite of the fact that this type is commonly used, little has been written about its characteristics. The writer's interest in this type is further augmented by the extended application of its characteristics, even beyond the beams that actually belong to this category. Any type of sectional variation may be viewed as a limiting condition of a formation of a large number of prismatic segments and therefore may be treated as a beam of this type. Indeed, it is a matter of common practice, in an approximate analysis of a beam of any type of sectional variation, to divide the beam into a number of parts and to treat each part as a prismatic segment, the degree of accuracy of such an approximation depending on the number of segments assumed.

The method under discussion is not only applicable to this type of beam, but its application here is particularly easy and adaptable. The use of this method for beams of prismatic segments was advocated[16] by the writer in 1947. The writer designated the momental areas A, A_1, and A_2, respectively, by a, m, and i, since these momental areas can be readily recognized as the area, moment, and moment of inertia of the analogous column. Subscripts A and B were used to indicate the particular point of reference, A and B being the ends of the span under consideration. The writer also used the symbol j to denote the momental area A_3.

In an approximate analysis, the loading, too, may be approximated with a sufficient degree of accuracy as a series of concentrated loads. Under such a treatment only the commonly familiar momental areas A, A_1, and A_2 are involved. However, since the determination of A_3 is so easy in the case of prismatic segments,[16] the writer could not resist the temptation of introducing it as j in a direct treatment of uniform loads. Incidentally, the direct treatment of uniform loads afforded the writer the opportunity of introducing the case of a uniform load on a symmetrical beam with its exceeding simplicity.

In his generalized discussion of the subject, the author used six cases of loading. Case 1 is a point loading, cases 2, 3, and 4 are area loadings of constant intensity, and cases 5 and 6 are area loadings of which the intensity is a linear function of the distance along the span. For a more complete generalization another area loading could have been introduced, the intensity of which is an nth degree function of the distance along the span. In addition to its theoretical generalization, such a loading would represent the weight of the beam itself. This loading, however, would require the use of momental areas

[16] "Design Constants Facilitate Analysis of Beams Composed of Prismatic Segments," by Harris Solman, *Civil Engineering*, May, 1947, p. 43.

of a degree up to and including $n + 3$. It is agreed that the additional complexity would not be warranted by the advantage gained by this generalization.

It may be noted that the author recognizes the existence of three "shape constants" for the general condition, whereas the writer[16] derived formulas for two constants at each end of the beam (carry-over and stiffness factors), making a total of four constants for the beam. This apparent discrepancy is not to be interpreted as a disagreement, as no attempt has been made in the writer's paper to eliminate redundancy. Indeed, the well-known relationship between the constants, $r_A K_A = r_B K_B$, reduces the number of independent constants to three, thus bearing out the author's statement.

Corrections for *Transactions:* In September, 1948, *Proceedings*, on page 1136, in Fig. 4, change "$l - k$" to "$1 - k$" in two places; and on page 1137, change Eq. 29 to read "$M' = M_1 - M_2 = -(x - k\,L)P$ and in Eq. 33*b*, change "$A_{1(kn)}$" to "$A_{(kn)}$."

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DISCUSSIONS

EXPANSION JOINT PRACTICE IN HIGHWAY CONSTRUCTION

Discussion

BY FRANK H. GARDNER

FRANK H. GARDNER,[13] ASSOC. M. ASCE.—This excellent treatment of spacing of expansion joints in concrete pavements stops short, however, of the total range of the subject in that the effect of subgrade friction on slab movement; reinforcing; formation of joints; load transmission; joint materials; prevention of infiltration; and other factors are not mentioned. It is hoped that in his closure the author will discuss these fields so that the paper will fully cover this important subject.

The writer agrees with the author's principal conclusions and predicts that the trends, in addition to those set out in the paper, will include:

1. Development of machines for cutting contraction joints after the concrete has set, or other methods to avoid interrupting the finishing operation at joints;
2. Increase of the tie across contraction joints by lapping the reinforcement at joints or, preferably, by additional corrosion resisting metal;
3. Omission of load transfer devices at joints so tied;
4. Improvement of joint filling compounds, possibly by the addition or use of recently developed plastics;
5. More extensive subsealing in areas adjacent to joints; and
6. Lengthening slabs to or beyond the upper limits suggested by the author.

The author is to be commended for his excellent treatment of the part of the subject covered.

NOTE.—This paper by A. A. Anderson was published in September, 1948, *Proceedings*.

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